APPLICATION OF RECIRCULATING INTERMITTENT SAND FILTERS TO IMPROVE SEPTIC TANK EFFLUENT QUALITY

Final Report
Prepared for
Ontario Ministry of the Environment
and
Great Lakes 2000 Sustainability Fund

Prepared by
A. Robertson
Research Associate
McMaster University

April 2002
Revised July 2002

MOE
APP
ANBI

FINANCIAL SUPPORT

This study was funded by the Ontario Ministry of the Environment (MOE) and Environment Canada. MOE's support was provided by the Drinking Water, Wastewater and Watershed Standards Section, Standards Development Branch. Environment Canada's support was provided by the Great Lakes 2000 Cleanup Fund (GL2000 CUF)

Ontario Ministry of the Environment

The Ontario Ministry of the Environment (MOE) supports scientific research to provide a better understanding of interactions between human activities and ecosystem health, as well as the development, demonstration and implementation of innovative and cost-effective technologies that maintain and enhance the natural environment. The Drinking Water, Wastewater and Watershed Standards Section has supported and/or worked co-operatively with federal and municipal governments and universities on numerous projects to advance the development of innovative approaches and technologies to provide more efficient water, sewage, and stormwater management services.

For more information regarding water, sewage, and stormwater optimization programs contact:

Manager
Drinking Water, Wastewater and Watershed Standards
Ministry of the Environment
Standards Development Branch
125 Resources Road
Etobicoke, Ontario M9P 3V6
Tel: (416) 327-7480
Fax:(416) 327-9091

Great Lakes 2000 Sustainability Fund

Environment Canada's Great Lakes 2000 Sustainability Fund (GLSF) supports development and implementation to restore beneficial use in Canada's 16 Area's of Concern. The Cleanup Fund has provided in excess of $60 million in support of more than 300 projects in the areas of sediment cleanup, combined sewer overflows, stormwater management, sewage treatment, and habitat rehabilitation.

For more information or copies of project reports contact:

Manager
Great Lakes 2000 Cleanup Fund
Environment Canada
867 Lakeshore Road,
Burlington, Ontario, L7R 4A6
Tel: (905) 336-6276
Fax: (905) 336-6272

DISCLAIMER

This report was prepared for the Ontario Ministry of the Environment, Environment Canada Great Lakes 2000 Cleanup Fund, and McMaster University as part of a cooperative project. The views and ideas expressed in this report are those of the authors and do not necessarily reflect the views and policies of the Ministry of the Environment, Environment Canada or McMaster University nor does the mention of trade names or commercial products constitute endorsement or recommendation for use.

ACKNOWLEDGEMENTS

This evaluation/demonstration study was sponsored by the Ontario Ministry of the Environment (MOE), and Environment Canada's Great Lakes 2000 Cleanup Fund (GL2000CUF), the Water Environment Association of Ontario (WEAO), Sand Filtration Inc. (SFI), Wilkinson Heavy Precast Ltd. and McMaster University. The study was endorsed by the Town of Minto, allowing the use of their Village of Clifford's sewage treatment plant, and by the Ontario Clean Water Agency (OCWA) which operates the plant.

We would like to acknowledge the assistance and cooperation of Larry Adams of the Village of Clifford, and Jon Planck, Paulette Hache, Omar Faruk, Al Brown, and Catherine Blott of SFI. Omar and Al assisted in the design of the pilot facility and Catherine in the actual construction. Bill Van Zutphen and Aaron Chase of OCWA have both provided on going assistance and in addition, Aaron has undertaken the field analyses and sample collection. We are indebted to David Turner of the Concrete Precasters Association and Jim Wilkinson and John Tran of Wilkinson Heavy Precast Ltd. for the supply of the concrete tankage. Tony Ho and Mano Manoharan of the MOE, Sandra Kok and Sandra Skog of Environment Canada, in particular, and all the members of the Technical Steering Committee have provided encouragement, direction, and an ongoing critique of the study as it proceeds.

Lastly, the contribution of advice and the support of the members of the Technical Steering Committee contributed greatly to the project. Members included:

Tony Ho	MOE, Standards Development Branch
Mano Manoharan	MOE, Standards Development Branch
Janusz Budziakowski	MOE, Environmental Assessment & Approvals Branch
Al Thachuk	MOE, West Central Region
Doug Huber	MOE, Southwestern Region
Sandra Kok	Environment Canada, GL2000 SF
Sandra Skog	Environment Canada, GL2000 SF
Larry Adams	Town of Minto
Keith Murphy	McMaster University
Anna Robertson	McMaster University
Glenn Caverly	Concrete Precasters Association (Executive Director)
Dave Turner	Concrete Precasters Association (Acton Precast)
Jim Wilkinson	Concrete Precasters Association (Wilkinson Heavy Precast)
John Tran	Concrete Precasters Association (Wilkinson Heavy Precast)
Peter Rupcic	OCWA (Client Service)
Bill Van Zutphen	OCWA (Plant Operations)
Jon Planck	Sand Filtration Inc.
Paulette Hache	Sand Filtration Inc.
Bob Crane	WEAO (Hamilton-Wentworth Region)
Alex Campbell	WEAO (R.J. Burnside)

EXECUTIVE SUMMARY

Currently, in Ontario, there are over 400 wastewater treatment plants. Approximately one hundred and fifty of these plants are lagoon treatment systems (conventional facultative lagoons). These lagoons are subject to significant seasonal effects which result in poorer effluent quality. In some cases, the systems have difficulty meeting in effluent criteria required by their certificate of approvals. Additionally, approximately 20% of Ontario's population lives in small communities and uses private septic tanks/tile beds for wastewater disposal. Many of these systems are aging or have not been maintained properly, resulting in system failure and contamination of surface and groundwater. As well, septic systems require substantial area. This reduces the population density and increases the consumption of arable land as the population increases. With the current pressures on rural communities to upgrade and expand their treatment facilities to satisfy public demand for better effluent quality, alternative treatment methods merited further investigation. The options of constructing either long sewers to existing treatment facilities in neighbouring communities, or new treatment plants are costly; for example, in 1997/1998, Ministry of the Environment (MOE) approved $16 million for 7 communities to remediate septic tank and tile field systems.

Based on a review of methods for upgrading lagoon effluents in Ontario (R.V. Anderson, 1992), intermittent sand filters (ISFs) were identified as a cost effective treatment method. ISFs, as used in Ontario, usually consist of a sand bed with a depth of approximately 1 m, a distribution system and a collection system with discharge to surface water. The sand has an effective diameter (D_{10}) of 0.10–0.15 mm. The lagoon effluent is applied daily, sometimes reaching depths of 30 cm over the filter, and allowed to percolate through the filter. Since 1981, ISFs have been used in Ontario to treat municipal wastewater following pretreatment in lagoon systems, but these are restricted to warm weather operation.

In 1996, McMaster University completed a study for MOE and the Great Lakes Sustainability Fund (GLSF) (then the Great Lakes Cleanup Fund) to assess the effectiveness of ISFs to upgrade the quality and reduce the potential toxicity of municipal lagoon effluent. Specifically, the removal of ammonia and phosphorus were investigated at the wastewater facility (facultative lagoons with alum addition followed by ISFs and wetland discharge) at New Hamburg, Ontario. The study results indicated that the effluent median and 75th percentile, over and 8 month period (April-November 1991), for total suspended solids (TSS) and 5-day biochemical oxygen demand (BOD_5) were 1.0 and 2.3 mg TSS/L and 1.9 and 3.2 mg BOD_5 /L respectively. Further, a consistent reduction in ammonia nitrogen was observed, even during cold weather operation. The median effluent nitrogen was 0.2 mg/L. At the site studied, the ISF influent was a relatively high quality, seasonal discharge lagoon effluent with low TSS and BOD_5. The median and 75th percentile were 8.8 and 15.7 mg TSS/L and 10.4 and 12 mg BOD_5/L. Ammonia nitrogen concentrations varied seasonally, with a low of 0.5 mg/L in the summer and a high of 13.6 mg/L in early spring and late fall.

In 1998, a literature review and site visits to selected Wisconsin sand filters was conducted to determine whether these filters could

- Treat influents with higher concentrations of TSS and BOD_5 such as effluents from aerated lagoons, continuous discharge lagoons and septic tanks,

- Operate on a year round basis in Southern Ontario,
- Be comparable in cost to seasonal discharge lagoon systems.

The study's aim was to also identify design and operating parameters necessary for optimal performance.

Various types of sand filters were evaluated. These were: conventional surface sand filters after lagoon pretreatment, recirculating surface sand filters (RSF) after septic tank pretreatment, and recirculating buried sand filters after septic tank pretreatment. In RSFs, the wastewater that has been pretreated in a septic tank is combined with a portion of the sand filter effluent before it is distributed to the sand filter and eventually discharged to a receiving water.

Media size used in Wisconsin ranged from 0.13 to 6.0-9.5 mm effective size. It was found that the treatment obtained by both ISFs and RSFs was independent of hydraulic loading rate (0.2 to 1.0 m/day, including recycle) but was correlated to the filter influent TSS or BOD_5 concentration. Recycle of treated effluent dilutes the incoming influent, reducing the influent concentrations. This resulted in substantial reductions in both the effluent TSS and BOD_5. Almost complete nitrification has been documented in this study for RSFs operating during the winter period. There was also some data that indicated appreciable denitrification (removal of nitrate) as well. There was substantial difference in the duration of the filter run time (less than 50 days to over 100 days), depending on quality of the influent applied to the filter. The capital costs associated with sand filters were comparable to treatment using aerated lagoons; however the RSF produces a better quality effluent. Capital costs for septic tank/RSF would be approximately two thirds the cost of the seasonal facultative lagoon/ISF.

From the above evaluation of sand filters, it was concluded that there was room for further optimization of RSFs to provide even more cost effective treatment. The areas recommended for further study were:

- Selection of media size,
- Selection of dosing frequency,
- Selection of appropriate quantity of recycle, and
- Selection of hydraulic loading rate.

As a result of the preliminary study, an evaluation of RSFs was carried out from 1999 to 2001. This study was designed to demonstrate the applicability of RSFs for year round treatment of municipal wastewater under Ontario environmental conditions. The study was also to provide information on RSF optimization to designers and operators in the areas of effects of media size, wastewater loading rate, recycle ratio, application frequency, feasibility of attaining complete or partial nitrification and denitrification. Finally, the study was also to provide a comparison of the performance of conventional RSFs to and MOE approved commercial unit (Orenco, Sutherlin, Oregon).

The pilot scale facility was constructed at the Town of Minto's STP in the Village of Clifford and consisted of four parallel $12m^2$ RSFs. Pretreatment for each filter was provided by a two-stage septic tank. A microcomputer control panel with telephone hook-up allowed frequent monitoring, resetting of parameters, and data logging of operational data. One of the 4 pilot scale filters was constructed with media (sand) typical

of municipal ISFs in Ontario while the other three were constructed with a considerably coarser media.

The data presented covers the start up and maturation of the four pilot scale RSFs during September to December 1999, followed by four phases covering December 1999 to May 2001. During start up (Acclimation Phase), the filters showed the expected removals for TSS and $CBOD_5$ (carbonaceous biochemical oxygen demand). Effluent values obtained ranged from 8-25 mg $CBOD_5$/L and 8-13 mg TSS/L. Since nitrifiers grow more slowly than heterotrophic bacteria, it was expected that nitrification would be much slower in commencing. This was borne out by effluent values of ammonia ranging from 20-28 mg nitrogen/L.

During the first winter phase (Phase #1), the effluent quality for the coarse media RSFs was consistently equivalent to typical municipal biological treatment plants. The $CBOD_5$ averaged from 5-19 mg/L and TSS was 14-19 mg/L. Despite filter effluent temperatures in the 4-6^{0}C range, the RSFs produced a partially nitrified effluent. Based on nitrogen balances, some denitrification also occurred. Effluent with 4-8 mg/l of total ammonia nitrogen and 4-15 mg/l of nitrate nitrogen was produced from a raw wastewater averaging 25 mg/l of total ammonia and 44 mg/L of total nitrogen for filters 1, 2 and 3. Filter 4 (fine media) did not nitrify as readily, and the average values for ammonia nitrogen and nitrate nitrogen were 25 and 1.2 mg N/L respectively. Filter #1 was run with twice daily dosing, rather hourly for part of this phase. Although the wastewater loading was 4 times the design value, the $CBOD_5$ and TSS still were within the range of values for the other filters (16 mg $CBOD_5$/L and 18 mg TSS/L). There was much less nitrification in this filter, with effluent ammonia of 17 mg N/L and nitrate of 1.2 mg N/L.

For Phase #2, the wastewater loading was doubled; the values of effluent $CBOD_5$ and TSS improved over the previous phase. For filters 1, 2 and 3, the average $CBOD_5$ ranged from 1.5 to 2.7 mg/L and the TSS averaged 10-13.2 mg/L. Filter #4 experienced significant hydraulic problems during this phase, with the effluent $CBOD_5$ ranging from 0.2 to 23.8 mg/L and TSS from 9.9-21.5 mg/L. Both nitrification and denitrification occurred to some degree in the first three filters. The effluent ammonia nitrogen and nitrate nitrogen averaged from 0.5-3.3 mg N/L and 5.0-13.6 mg N/L respectively, with the total nitrogen (TN) decreasing from 28 mg/L in the primary tank effluent to an average of 10-15 in the filter effluents. Filter 4 still experienced difficulty nitrifying at the beginning of this phase, but over the latter part of the phase, and with reduced flow, nitrification commenced. The values for TN, ammonia nitrogen and nitrate nitrogen were 7.7-24.3, 3.4-16.7 and 1.3-6.5 mg N/L respectively.

The wastewater flow to filter #1 was reduced for Phase #3, with flow to the others remaining the same. The $CBOD_5$ and TSS values for all four filters averaged from 2.2-4.2 mg COD_5/L and 5.7-10.9 mg TSS/L. All filters achieved almost complete nitrification and varying degrees of denitrification were observed. The average values were 0.8-5.3 mg/L ammonia nitrogen, 6.3-14 mg/L nitrate nitrogen and 10.7-17.4 mg/L total nitrogen.

For the final phase, the flows remained the same to each filter. Since this was a winter phase, and filter #4 was still experiencing hydraulic difficulties, it was shut down before this phase commenced. Over this phase, the effluent temperatures averaged 5-6°C. The filters continued to operate under cold ambient conditions, producing high quality effluent, although with some reduction in nitrification. $CBOD_5$ and TSS were 2.9-6

mg/L and 4.9-9.7 mg/L respectively. Effluent ammonia ranged from 0.8 to 13.3 mg N/L, nitrate 5.6-14.4 mg N/L and TN 10.6-19.8 mg N/L, down from 31.8 mg N/L in the primary tank effluent.

These results indicate that wastewater may be successfully treated year round under Ontario environmental condition using septic tank followed by a recirculating intermittent sand filter using the coarser media.

TABLE OF CONTENTS

TABLE OF FIGURES

LIST OF TABLES

1.0 INTRODUCTION

Currently, in Ontario, there are over 400 wastewater treatment plants. Approximately one hundred and fifty of these plants are lagoon treatment systems (conventional facultative lagoons). These lagoons are subject to significant seasonal effects which result in poorer effluent quality. In some cases, the systems have difficulty meeting in effluent criteria required by their certificate of approvals. Additionally, approximately 20% of Ontario's population live in small communities and use private septic tanks/tile beds for wastewater disposal. Many of these systems are aging or have not been maintained properly, resulting in system failure and contamination of surface and groundwater. As well, septic systems require substantial area. This reduces the population density and increases the consumption of arable land as the population increases. With the current pressures on rural communities to upgrade and expand their treatment facilities to satisfy public demand for better effluent quality, alternative treatment methods merited further investigation. The options of constructing either long sewers to existing treatment facilities in neighbouring communities, or new treatment plants are costly; for example, in 1997/1998, MOE approved $16 million for 7 communities to remediate septic tank and tile field systems.

Based on a review of methods for upgrading lagoon effluents in Ontario (R.V. Anderson, 1992), intermittent sand filters (ISFs) were identified as a cost effective method. ISFs, as used in Ontario, usually consist of a sand bed with a depth of approximately 1 m, a distribution system and a collection system with discharge to surface water. The sand has an effective diameter (D_{10}) of 0.10–0.15 mm. The lagoon effluent is applied daily and allowed to percolate through the filter. Since 1981, intermittent sand filters (ISFs) have been used in Ontario to treat municipal wastewater following pretreatment in lagoon systems, but these are restricted to warm weather operation.
In 1996, McMaster University completed a study for MOE and the GLSF to assess the effectiveness of ISFs to upgrade the quality and reduce the potential toxicity of municipal lagoon effluent. Specifically, the removal of ammonia and phosphorus were investigated at the wastewater facility (facultative lagoons with alum addition followed by ISFs and wetland discharge) at New Hamburg, Ontario. The study results indicated that the effluent median and 75th percentile, over and 8 month period (April-November 1991), for total suspended solids (TSS) and 5-day biochemical oxygen demand (BOD_5) were 1.0 and 2.3 mg TSS/L and 1.9 and 3.2 mg BOD_5 /L respectively. Further, a consistent reduction in ammonia nitrogen was observed, even during cold weather operation. The median effluent nitrogen was 0.2 mg/L. At the site studied, the ISF influent was a relatively high quality, seasonal discharge lagoon effluent with low TSS and BOD_5. The median and 75th percentile were 8.8 and 15.7 mg TSS/L and 10.4 and 12 mg BOD_5/L. Ammonia nitrogen concentrations varied seasonally, with a low of 0.5 mg/L in the summer and a high of 13.6 mg/L in early spring and late fall.
In 1998, a literature review and site visits to selected Wisconsin sand filters was conducted to determine whether these filters could

- Treat influents with higher concentrations of TSS and BOD_5 such as effluents from aerated lagoons, continuous discharge lagoons and septic tanks,
- Operate on a year round basis in Southern Ontario,

- Be comparable in cost to seasonal discharge lagoon systems.

The study's aim was to also identify design and operating parameters necessary for optimal performance.

Various types of sand filters were evaluated. These were: conventional surface sand filters after lagoon pretreatment, recirculating surface sand filters (RSF) after septic tank pretreatment, and recirculating buried sand filters after septic tank pretreatment. In RSFs, the wastewater that has been pretreated in a septic tank is combined with a portion of the sand filter effluent before it is distributed to the sand filter and eventually discharged to a receiving water.

Media size used in Wisconsin ranged from 0.13 to 6.099.5 mm effective size. It was found that the treatment obtained by both ISFs and RSFs was independent of hydraulic loading rate (0.2 to 1.0 m/day, including recycle) but was correlated to the filter influent TSS or BOD_5 concentration. Recycle of treated effluent dilutes the incoming influent, reducing the influent concentrations. This resulted in substantial reductions in both the effluent TSS and BOD_5. Almost complete nitrification has been documented in this study for RSFs operating during the winter period. There was also some data that indicated appreciable denitrification (removal of nitrate) as well. There was substantial difference in the duration of the filter run time (less than 50 days to over 100 days), depending on quality of the influent applied to the filter. The capital costs associated with sand filters were comparable to treatment using aerated lagoons; however the RSF produces a better quality effluent. Capital costs for septic tank/RSF would be approximately two thirds the cost of the seasonal facultative lagoon/ISF.

From the above evaluation of sand filters, it was concluded that there was room for further optimization of RSFs to provide even more cost effective treatment. The areas recommended for further study were:

- Selection of media size,
- Selection of dosing frequency,
- Selection of appropriate quantity of recycle, and
- Selection of hydraulic loading rate.

As a result of the preliminary study, an evaluation of RSFs was carried out from 1999 to 2001. This study was designed to demonstrate the applicability of RSFs for year round treatment of municipal wastewater under Ontario environmental conditions. The study was also to provide information on RSF optimization to designers and operators in the areas of effects of media size, wastewater loading rate, recycle ratio, application frequency, feasibility of attaining complete or partial nitrification and denitrification. Finally, the study was also to provide a comparison of the performance of conventional RSFs to and MOE approved commercial unit (Orenco, Sutherlin, Oregon).

2.0 PROJECT COORDINATION

This study was conducted by McMaster University under the auspices of the Ontario Ministry of Environment in conjunction with the Great Lakes 2000 Sustainability Fund. A technical steering committee, comprised of representatives from government agencies, consulting, and private industry, provided direction and comments on the study. McMaster University performed the initial literature review and from that, developed the experimental design. Sand Filtration Inc., (SFI) was responsible for the detailed design, construction, commissioning and routine maintenance of the pilot facility. SFI also gathered the flow data that was used by McMaster in the analysis of the data. MOE provided laboratory support, with some analyses replicated at McMaster University. The final data analysis and report were the responsibility of McMaster, with guidance from MOE and GLSF.

3.0 PROCESS DESCRIPTION

The two basic types of intermittent sand filters are conventional and recirculating. With conventional ISFs the effluent from the pretreatment process is applied directly to the ISF and is discharged once it filters through the ISF. Application of pretreated wastewater usually occurs once each day and can reach a depth of 15-30 cm on the surface near the end of a filter run. Once this starts to occur, the filter is taken out of service and rehabilitated. A schematic drawing of and ISF is shown in Figure 3.1.

With the recirculating ISF (or RSF), the pretreated process effluent is mixed with a portion of the RSF effluent before being applied to the filter. The recycle of the treated effluent dilutes the applied TSS and BOD_5 by augmenting the flow. With the higher combined influent flow, there is normally more frequent application to the filter. Typically the number of applications to a RSF is increased from 1 or 2 per day to 1 or more per hour. Figure 3.2 shows a schematic of an RSF. Conventional or recirculating ISFs may be constructed at the surface or may be buried. Successful operation of an ISF depends upon maintaining aerobic conditions in the filter media. Oxygen may enter the filter media from the filter surface, if ponding of the applied influent is prevented, or from the filter underdrains if they are not surcharged. With a buried filter venting through the covering layer into the media is required.

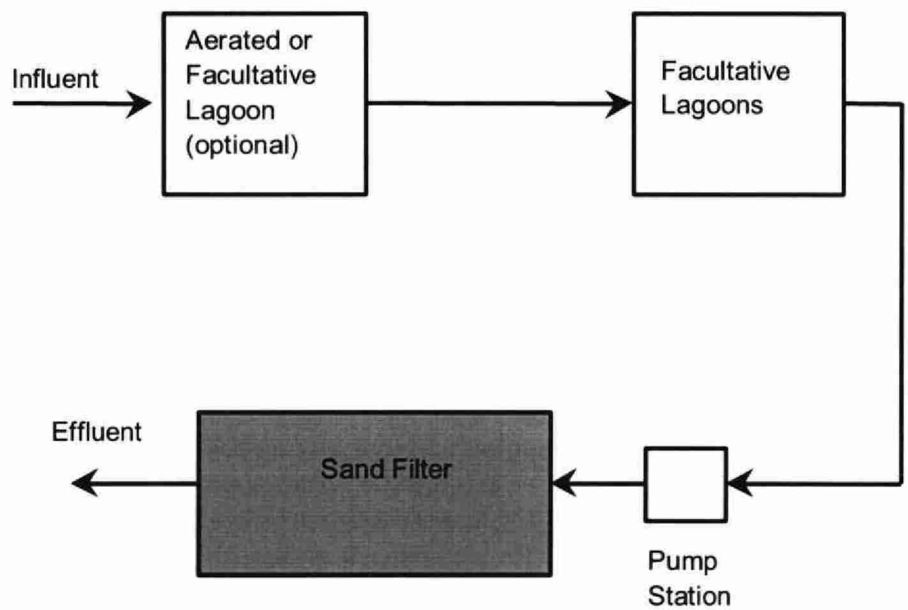

Figure 3.1: Conventional Intermittent Sand Filter

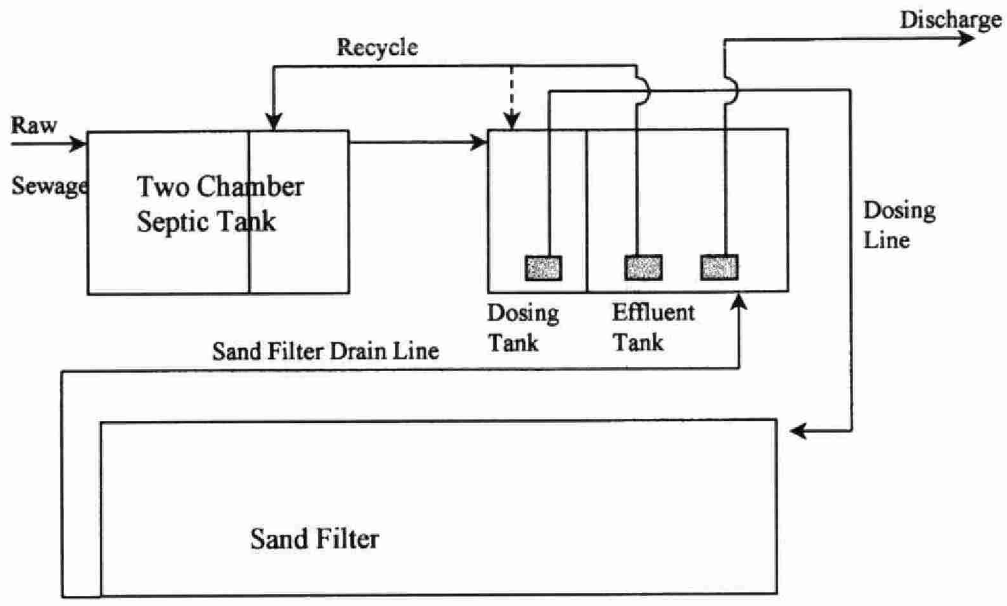

Figure 3.2: Recirculating Intermittent Sand Filter

4.0 PILOT PLANT DETAILS

The conceptual design (Figure 4.1), developed by McMaster University, called for four parallel 12m^2 RSFs. Pretreatment for each filter was to be provided by a two-stage septic tank with approximately a 2-day hydraulic retention. Rates of recirculation of RSF effluent to the second stage septic tanks would be an operational variable. One of the 4 pilot scale filters was constructed employing media typical of the size used by the municipal ISFs in Ontario (effective size ~0.1 mm.) while the other three were constructed with media with an effective size of ~2.6 mm. This was typical both of recent RSFs constructed in Wisconsin and the Orenco RSF. The wastewater was pumped from the main wet well to a primary septic tank. From the primary tank, the wastewater flowed by gravity to a pump tank. For research purposes, the primary septic tank chamber was separated from the secondary septic tank chamber by a pump chamber. This allowed for variability in the experimental design while still maintaining a common influent to all four filters. The wastewater was then pumped to the secondary septic tank chamber. For the generic RSFs, the wastewater was then mixed with recycled filter effluent. For the Orenco RSF, the filter effluent was recycled to the dosing tank instead of the secondary chamber of the septic tank. From the secondary septic tank chamber, it flowed by gravity to a dosing tank. There was a solids screen present in the doing tank of the Orenco RSF, but not in the generic systems. Filters #1 and #4 were equipped with both daily and hourly dosing tanks, while filters #2 and #3 only had the hourly dosing tanks. From the dosing tanks, the wastewater and recycled effluent was applied to the sand filter. Figure 4.4 shows a filter with the wastewater being applied to the surface of the filter. The filter effluent was collected in underdrains and flowed to a splitter from where a predetermined fraction was returned to the secondary septic chamber or dosing tank and the remainder flowed to a wet well and then was returned to the main wet well. Since the flow to the pilot facility was less than 5% of the total flow of the whole facility, significant dilution of the wastewater by this operation did not occur.

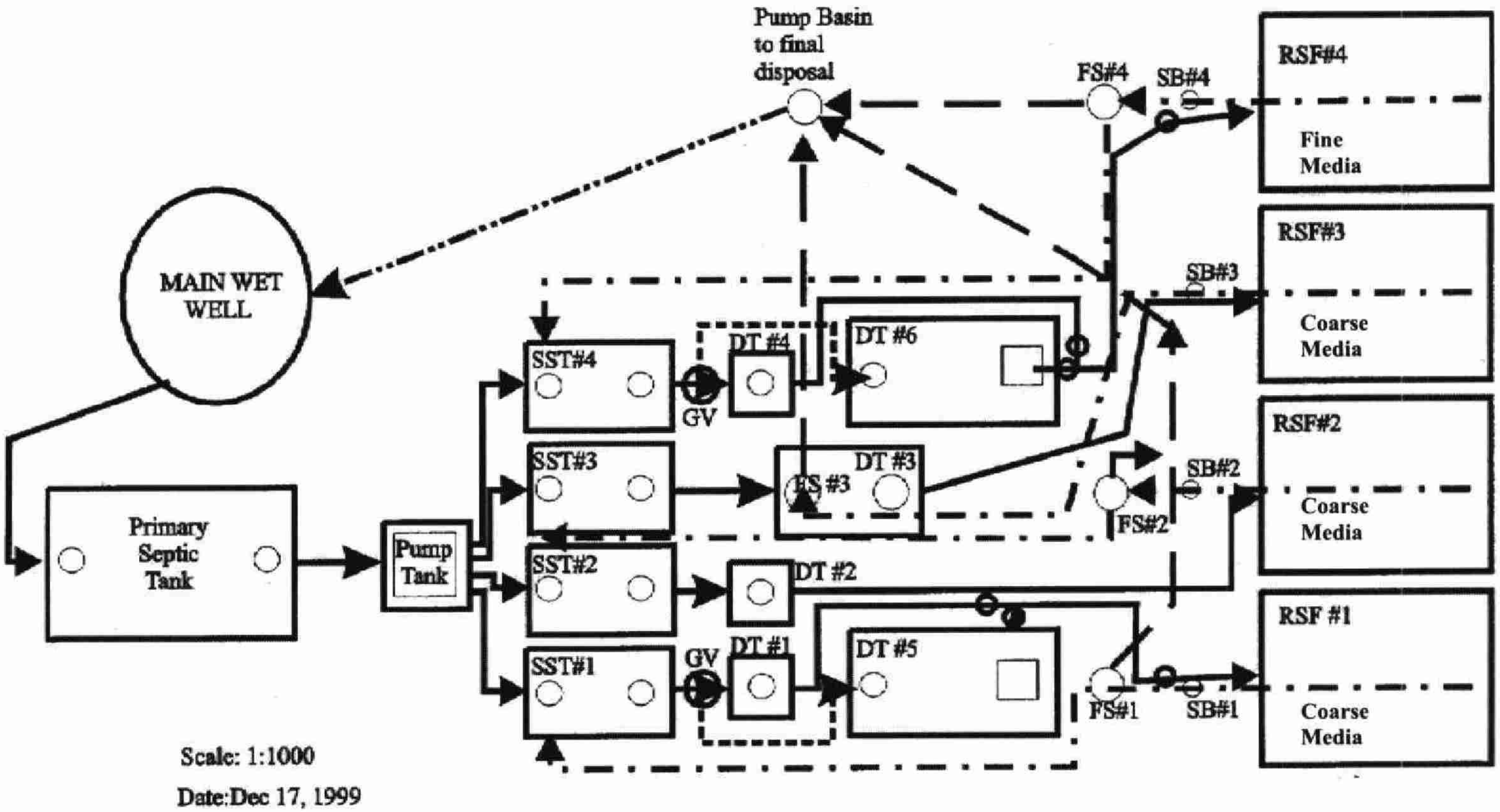

Figure 4.1: Clifford Sand Filter Research Facility General Layout

4.1 Construction and Operation

The pilot facility was constructed at Clifford during the period of June to August of 1999. The pilot facility parameters for tankage and filters are provided in Table 4.1. Sand Filtration Inc. (SFI) based on a process design developed jointly by McMaster University and SFI undertook the actual design and construction of the pilot plant. Wilkinson Heavy Precast Ltd., through the Concrete Precasters Association of Ontario (CPA), designed and supplied the precast concrete tanks used for the pretreatment septic tanks, pump tanks and dosing tanks in the pilot facility. The sand filter was constructed with plywood walls and a sand base. The filter base and walls were covered with a 30-mil PVC liner. The depth of media was designed to be 0.9-1.0m, but a high water table at the site restricted the depth to 0.61m. A three week period of equipment testing and calibration followed the construction, with the commissioning of the pilot facility on September 2, 1999. Data logging commenced on September 13, 1999. The experimental design called for all four RSFs to be operated initially at a wastewater flow of 0.2 m/day with a recycle ratio of 4:1. This was recommended in the US EPA Design Manual for Onsite Treatment and Disposal (1980). Figure 4.2 shows an overview of the site, Figure 4.3 shows an individual sand filter with coarse media and Figure 4.4 shows an individual filter with fine media.

The experimental system is controlled by a custom commercial control panel supplied by Orenco. The digital control consists of a microprocessor and the associated circuitry to allow communication over a standard telephone line, digital outputs for the control of relays or other on/off events, digital inputs to sense switch closures or other Orenco inputs, and finally, analog inputs to sense the analog outputs from pressure transducers or other analog devices. A detailed description of the control circuit and operation is presented in Appendix I.

Table 4.1 Clifford Pilot Scale RSF Design Data

Component	Detail	Experimental Stream			
		1	2	3	4
Main Pump Well	**Pump**	SE150			
	Operating flow (calibrated) L/min	144.60			
Septic Tank	Primary Tank Operating Volume, L	11300			
Pump Tank	Pump Tank Operating Volume, L	Included in First Stage Operating Volume	Included in First Stage Operating Volume	Included in First Stage Operating Volume	Included in First Stage Operating Volume
	Pump	PEF33	PEF33	PEF33	PEF33
	Operating flow (calibrated) L/min	182.40	181.20	180.96	177.36
Secondary Septic Tank	Second Stage Tank Operating Volume, L (SST)	3925	3925	3925	3925
Recirculating Tank	Operating Volume, L	-	-	2700	-
	Pump			P2005	
	Operating flow(calibrated) L/min			99.37	
Dosing tank	Operating Volume, L(hourly dosing)	1204	1204	-	1204
	Operating Volume, L(daily dosing)	7356	-	-	7356
	Pump	PEF33E	PEF33E	-	PEF33E
	Operating flow(calibrated) L/min	42.4	46.8	-	47.7
Sand Filter	No. Of Laterals per Cell	2	2	5	2
	Diameter of Laterals, mm	32	32	25	32
	Distance between Laterals, mm	1500	1500	600	1500
	Length of Laterals, mm	3750	3750	3350	3750
	No. Of Orifices	16	16	35	16
	Orifice location	side	side	bottom	side
	Orifice diameter, mm	6	6	3	6
	Orifice spacing, mm	1250	1250	600	1250
Distribution Media	Depth	0	0	100	0
	Type - gravel (mm)	none	none	6 pea	none
Treatment Media	Surface Area, m^2	11.8	12.0	12.0	11.8
	Depth, mm	610	610	610	610
	Effective Size, mm	2.6	2.6	2.6	1st:0.10 2nd:0.14
	Uniformity Coefficient	2.0	2.0	2.0	1st:2.5 2nd:6.4
Collection Media	Depth, mm	230	230	230	230
	Type-gravel (mm)	10	10	10	10
Underdrains	Number (per filter)	1	1	1	1
	Depth of 1.5 to 2.0 mm gravel over underdrain pipe(mm)	150	150	150	150
	Diameter of Underdrains, mm	100	100	100	100
	Vents per Filter	2	2	0	2
Recirculation Device	Type	Splitter basin	Splitter basin	Orenco recirculating splitter valve RSV3U	Splitter basin

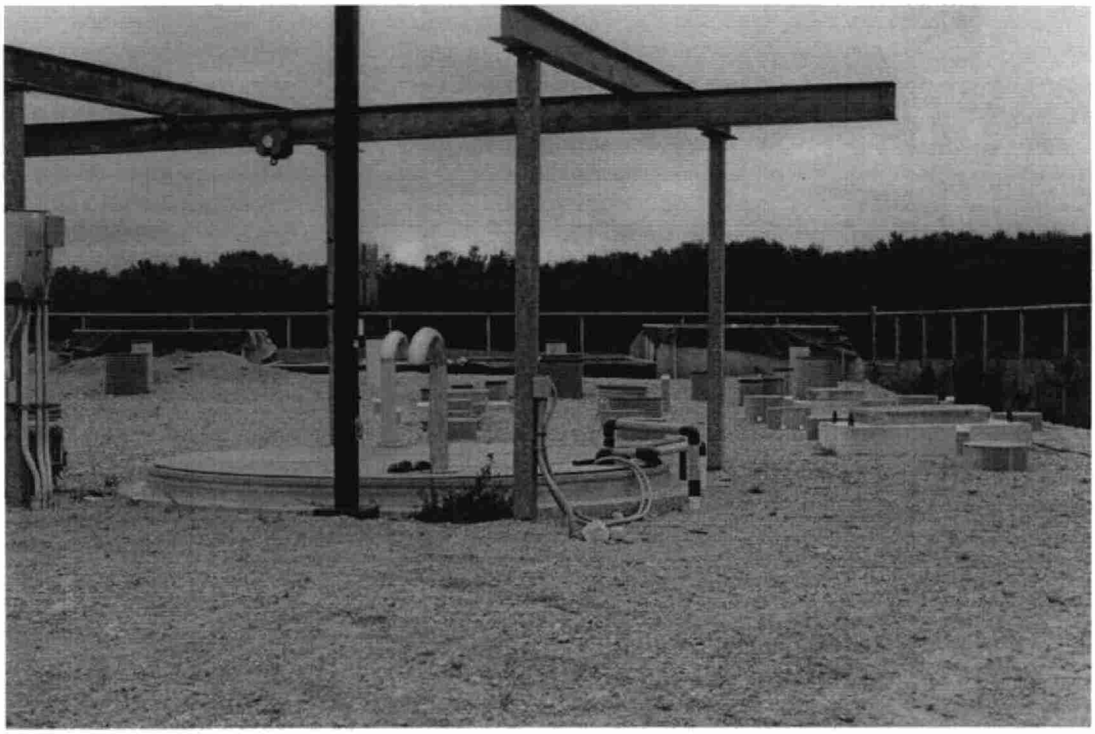

Figure 4.2: Clifford Pilot Plant Site

Figure 4.3: Individual Filter Showing Coarse Sand and Wastewater Application

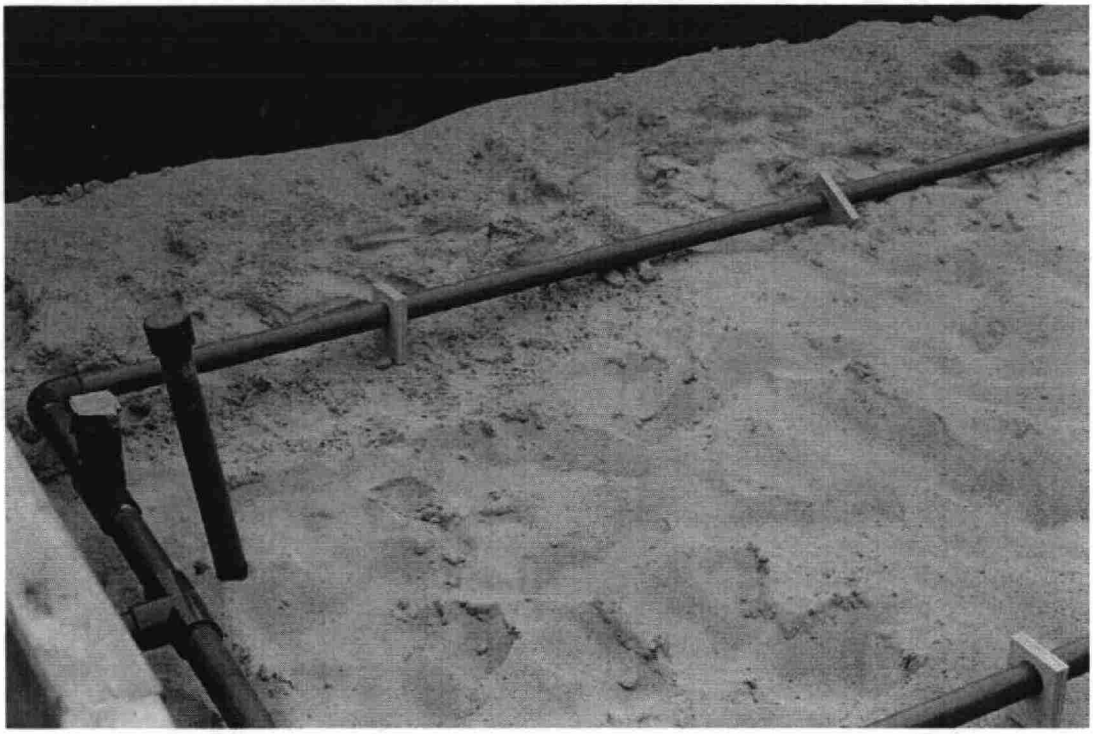

Figure 4.4: Individual Filter Showing Fine Sand

4.2 Sampling and Analysis

Existing Clifford plant data for two years prior to and during the study were tabulated and are reported in Table 4.2. These values are averages of twice monthly grab samples taken by the operator. These values showed that the wastewater used in this study was typical of a municipal wastewater.

During the study, grab samples were collected manually, normally on a weekly basis, to characterize the raw wastewater, the primary septic tank effluent, and each of the RSF influents and effluents. Analysis of the collected samples was performed by the MOE Central Laboratory. Additional analysis of selected parameters, on split samples, was performed by McMaster University to provide for ongoing process evaluation and control. The list of analyses and the results of the split sampling by the MOE Central Laboratory and McMaster University are presented in Appendix I together with the analytical methods used by both laboratories.

Table 4.2: Analysis of Clifford Wastewater

Year	Flow (m^3/d)	$CBOD_5$ (mg/L)	TSS (mg/L)	TKN (mg N/L)	NH_3+NH_4 (mg N/L)	TP (mg/L)
1997	136.6	142	165	32	26	4.8
1998	128.4	192	197	42	36	5.5
1999	132.4	147	175	44	37	5.6
2000	181.1	157	210	28	24	4.4
Minimum	128	81	107	15.8	14.4	1.94
Maximum	222	482	823	53.6	47.8	8.8

Equipment and manpower constraints required the use of grab samples rather composite samples. The use of grab samples can cause a greater variability from sample to sample, especially if the sampling chambers are small. Analyses performed on total samples as opposed to filtered samples had greater variability caused by the presence of solids. For both chemical oxygen demand and suspended solids, the results from McMaster were generally higher than the MOE results. The results for nitrate, ammonia, chloride and phosphate, which are performed on filtered samples, agreed very closely. Based on these considerations, the results between the two laboratories were considered to be within reasonable agreement.

4.3 Flow Balances

Daily flow data are available for the main pump, that pumps the wastewater into the primary septic tank, and for pumps SST#1, #2, #3, & #4, that divide the flow into the secondary septic tanks. Pump flows are determined from the daily run times converted to flows using pump calibration curves prepared by SFI for the individual pumps. Flow summaries were tabulated by SFI for RSFs #1, #2, #3, & #4 and for the Main Pump and forwarded to McMaster University. These summaries identified days and locations when it was suspected that incorrect data had been logged. The causes identified by SFI included power outages and malfunctioning sensors. Five operational periods have been identified within this overall period:

September 13, 1999 to November 11, 1999 - Acclimation Phase
November 29, 1999 to April 15, 2000 - Phase 1
May 12, 2000 to August 18, 2000 - Phase 2
September 22, 2000 to November 27, 2000 - Phase 3
December 21, 2000 to April 30, 2001 - Phase 4.

It should be noted that on days when the data were suspected to be erroneous they were deleted from the analysis. During Phase 3, the data for the main pump were calculated by SFI from the data available to them, and the Phase 4 numbers for the main pump are not available. The results are summarized in Table 4.3.

Table 4.3: Daily Flow Balance

Period	Daily Flow		Difference		
	Main Pump	SST# 1,2,3,4	Mean	%	Std. Dev.
	L/day	L/day	L/day		
Acclimation Phase 9/13/99-11/11/99	8029	8446	-903	-9.6	1422
Phase 1 11/29/99-4/15/00	10432	11752	-1357	-22.1	2009
Phase 2 5/12/00-8/15/00	14596	13298	141	-1.25	3585
Phase 3 9/22/00-12/20/00	13317	15229	-1911	-14.35	1977
Phase 4 12/21/00-5/2/01		13607			

Considering the potential for errors, the results of the flow balance seem reasonable and would validate the flow split to the four RSFs.

The wastewater flow from the secondary septic tanks was augmented by recirculated RSF effluent prior to being pumped to the filters. The quantity of recirculated flow to each filter was determined by a flow splitter that was set for the recycle specified in the experimental design. Daily flow data are available for the four pumps DT#1, #2, #3, & #4 that apply the combined flow onto the intermittent sand filters. Again, pump flows are determined from the daily run times converted to flows using pump calibration curves prepared by SFI for the individual pumps. As there was no provision for measuring the recirculated flows, it was proposed that a chloride mass balance would be employed to verify the flows to the individual RSFs. Unfortunately, leakage of chloride from the concrete tanks throughout the entire study period negated this approach and it was not possible to verify the recirculated flows.

For this study, forward flow was defined as the undiluted wastewater flow applied to the RSF i.e. the amount of primary septic tank effluent that was pumped to the secondary septic tank. Total flow was the forward flow plus recycle flow that was applied to the RSF. The recycle ratio was defined as the (total flow-forward flow)/forward flow.

5.0 EXPERIMENTAL DESIGN

An experimental design was developed for the four pilot plant RSFs to permit a direct comparison of RSF performance under a range of design and loading parameters. Table 5.1 presents a summary of the parameters and ranges for each parameter that were considered. The sieve analyses for the coarse and fine sand that was used are presented in Appendix II.

Table 5.1: Experimental Design

Parameter	Range
Media Effective Size (mm)	0.1 – 2.6
Wastewater Hydraulic Loading (m/day)	0.2 – 0.4
RSF Effluent Recycle Ratio	2:1 – 4:1
TSS or BOD Loading (g/m^2-day)	40 - 80
Application Frequency (hours)	0.3 – 24
Operating Temperature (^{0}C)	5 - 20

The design also allowed a direct comparison of a generic RSF with the Orenco RSF under all operating conditions.

6.0 PILOT PLANT PERFORMANCE

Almost immediately following start up the Primary Septic Tank provided a substantial amount of pretreatment (Figure 6.1). Over the entire study, total suspended solids (TSS) was reduced by approximately 80 % and carbonaceous biochemical oxygen demand ($CBOD_5$) by 45 %.
Figure 6.2 indicates that the variability of the effluent total ammonia, ranging from 14 to 32 mg/L, was less than the influent (7 to 55 mg/L). Despite a reduction in variability, no overall removal appeared to occur. In Figure 6.3, it can be seen that there is about 30 % TKN removal in the Primary Tank.
For each study phase, the average values for the parameters shown on the following graphs are given in the Tables 6.1-6.5.

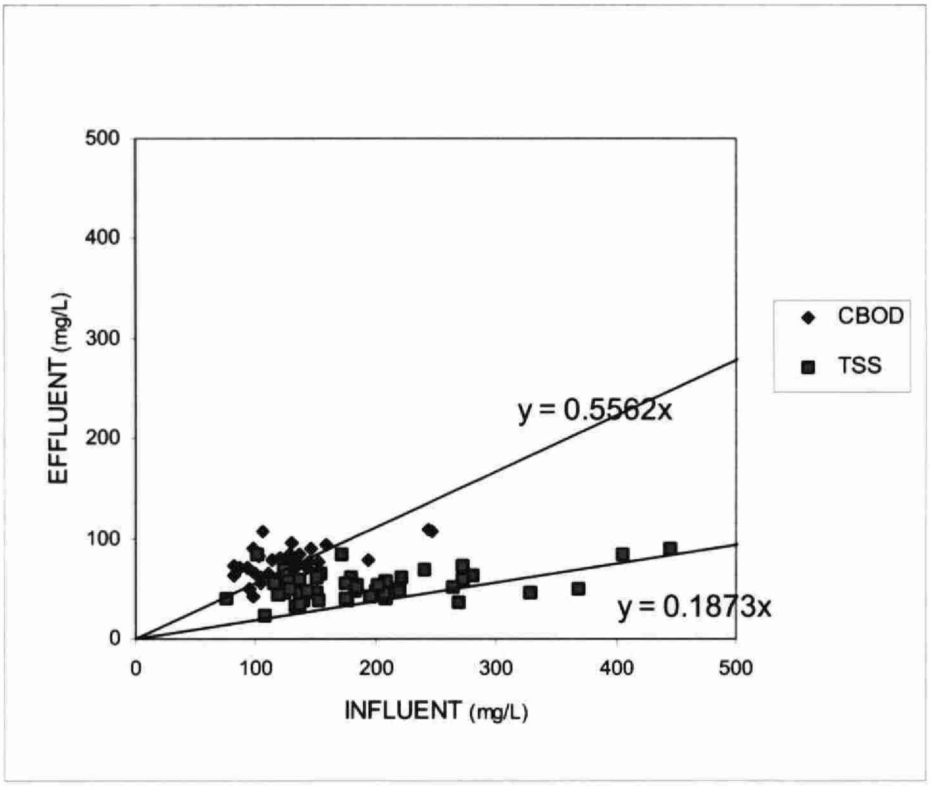

Figure 6.1: TSS and $CBOD_5$ Removal in Primary Septic Tank September 1999-May 2001

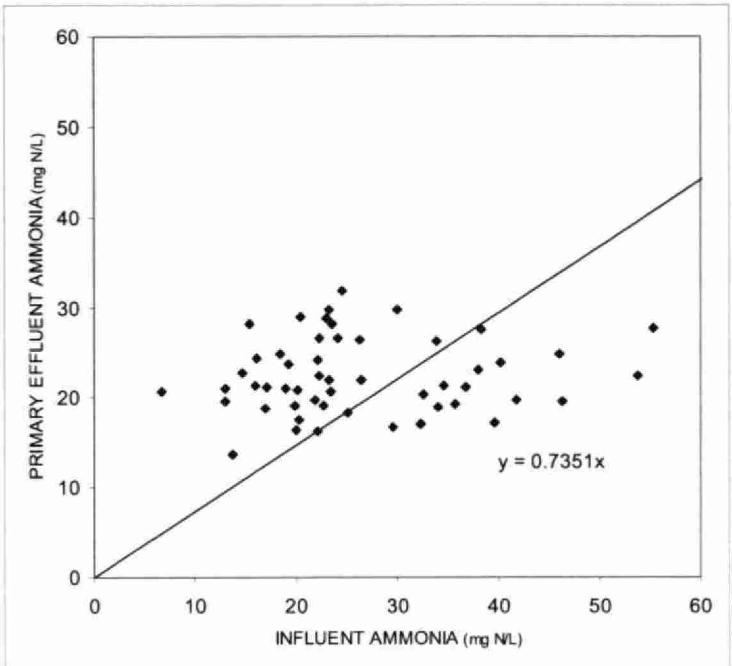

Figure 6.2: Primary Tank Ammonia Removal, September1999-May 2001

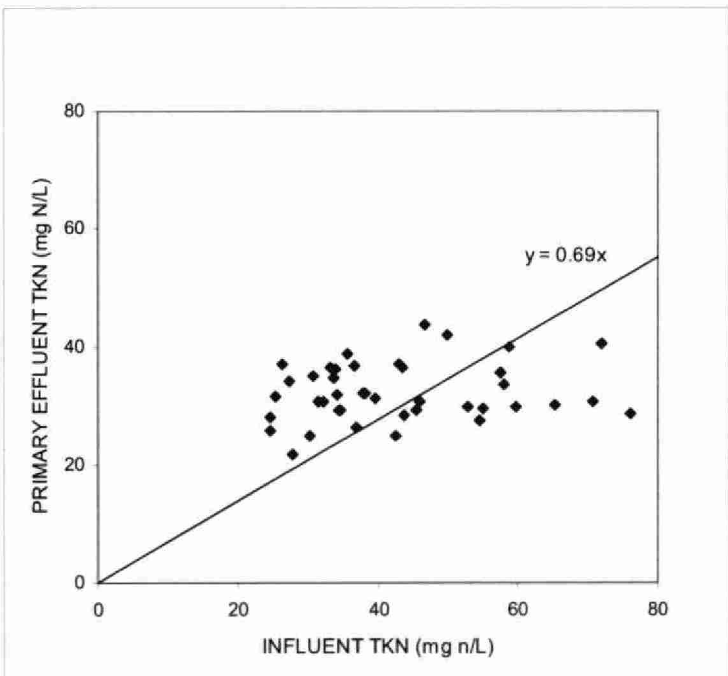

Figure 6.3: Primary Tank TKN Removal, September 1999-May 2001

6.1 Acclimation Phase

The RSF performance during the initial start up period, presented in Table 6.1, demonstrated the TSS and $CBOD_5$ removal capabilities of the RSF treatment system. As would be anticipated with the slow growth of nitrifiers, no appreciable nitrification occurred (Figure 6.4) nor was there any appreciable removal of nitrogen after the primary tank (Figure 6.5). The nitrogen removal takes into consideration individual filter streams but does not include removal in the Primary Tank. RSF #4 was taken out of service on October 2nd because of surface ponding. The performance of the other three RSFs deteriorated in early November caused by clogging of the geotextile cloth placed over the RSFs underdrains. Effluent $CBOD_5$ values increased to 26-33 mg/l. There was considerable variation in both wastewater flow and recycle flow to the filters; the average flow numbers for this phase are given in Table 6.1 and flow data for all phases is provided in Appendix IV. The E.Coli population decreased by about two orders of magnitude from the influent to the filter effluent.

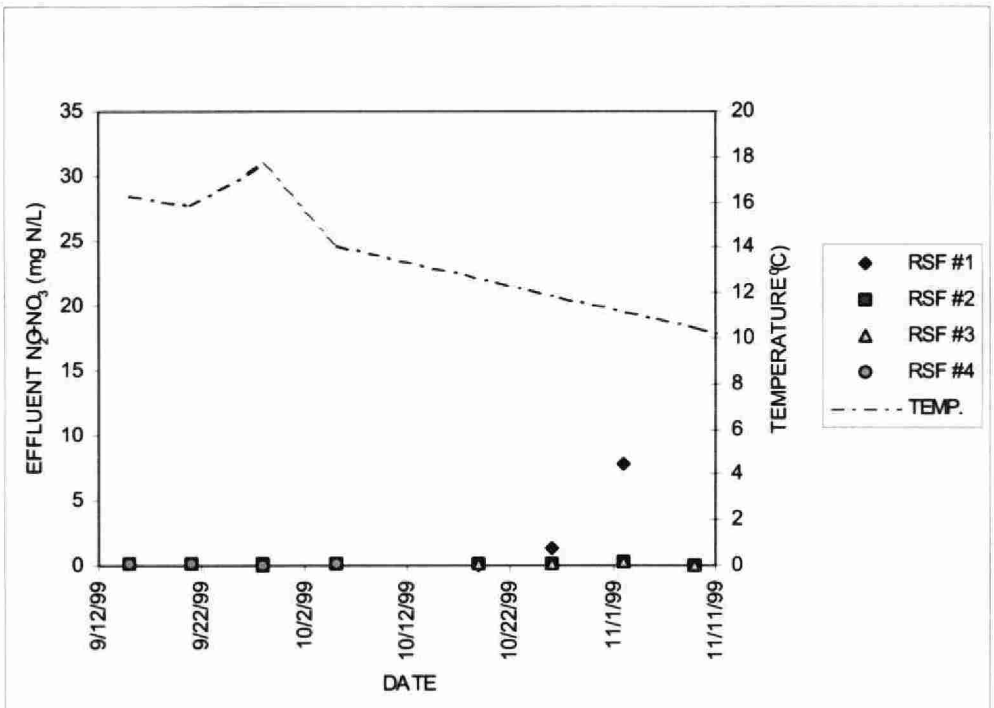

Figure 6.4: Acclimation Phase Ammonia Conversion

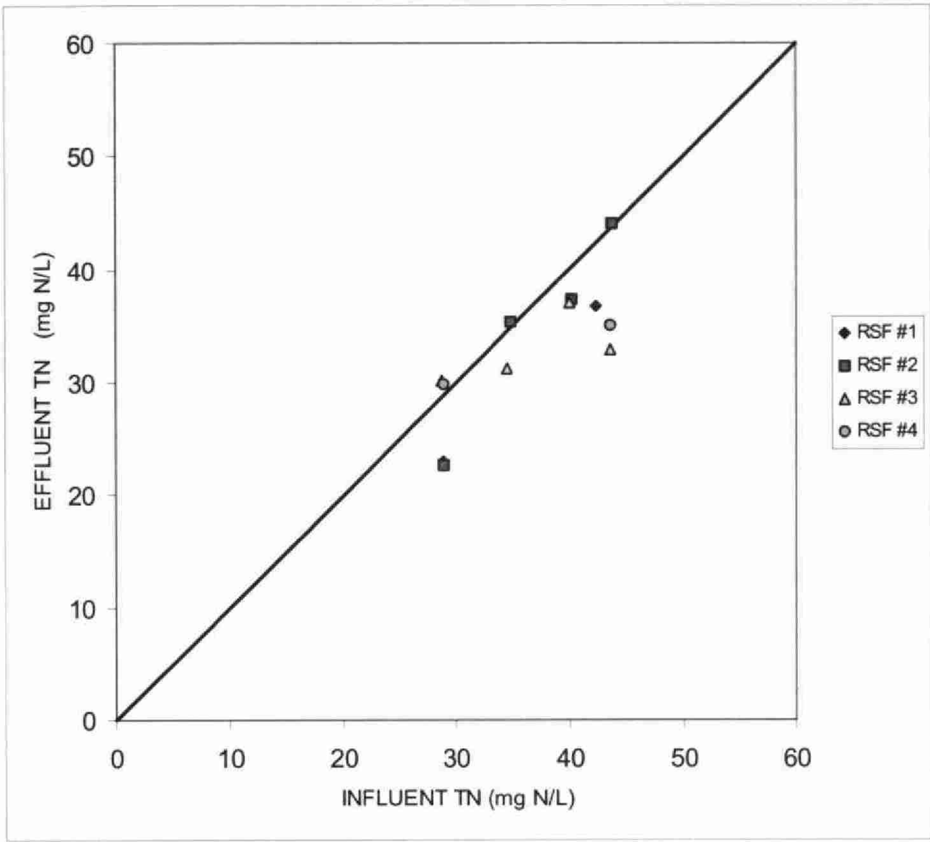

Figure 6.5: Acclimation Phase Nitrogen Removal

Table 6.1: Acclimation Phase Results (September, 1999 –November, 1999)

	WASTEWATER	PRIMARY TANK EFFLUENT	RSF #1 EFFLUENT	RSF #2 EFFLUENT	RSF #3 EFFLUENT	RSF #4 EFFLUENT
DURATION OF FLOW			9/13 –11/1	9/13 –11/1	9/13 –11/1	9/13 –10/1
MEDIA SIZE (mm)			2.6	2.6	2.6	0.1
UNIFORMITY COEFFICIENT			2	2	2	2.4
WASTEWATER FLOW RATE (m/d)						
DESIGN			0.20	0.20	0.20	0.20
ACTUAL			0.23	0.24	0.34	0.26
TOTAL FLOW RATE (m/d)						
DESIGN			1.0	1.0	1.0	1.0
ACTUAL			0.61	0.88	0.94	0.85
RECYCLE RATE						
DESIGN			4:1	4:1	4:1	4:1
ACTUAL			2.5:1	2.6:1	1.8:1	2.3:1
$CBOD_5$ (mg/L)	150	72	8	11	13	25
TSS (mg/L)	299	42	13	8	8	8
NH_3+NH_4 (mg N/L)	28.3	27.9	20	28	28	27
NO_2 (mg N/L)			1.1	0	0	0
NO_3 (mg N/L)			0.6	0.1	0.1	0.1
TN (mg N/L)	47.6		31.3	34.7	32.8	32.4
E.Coli (10^3counts/100mL)	12471	3957	681	900	833	1048

6.2 Phase 1

The RSFs were returned to service in late November to mid December, after the geotextile cloth had been removed. Ponding again caused the surface of RSF #4 to freeze on approximately January 7th. Starting on January 19th attempts were made to thaw RSF #4 and on Feb 2nd it was placed back in service. RSFs #1, #2 and #3 were covered by snow and ice during this period (Figure 6.6), but continued to provide high quality effluent.

Figure 6.6: Sand Filters During Winter Operation

Performance results for this phase are presented in Table 6.2. $CBOD_5$ and TSS removals were similar to the acclimation phase although there was some deterioration of quality for RSF #1, especially during the early part of this phase when twice daily dosing was used. At that time, RSF #1 was receiving 4 times the design wastewater flow. Once the flow was reduced to closer to design, the effluent quality started to improve. Nitrification, which had been negligible or non-existent in the RSFs during the Acclimation Phase, increased and by January 18th a partially nitrified effluent was produced (Figure 6.7), despite the consistent low effluent temperature of 3-6 ^{0}C in the operating RSFs. The temperature shown on the graph is an average of the temperatures for the four filters on any given day. It should be noted that the high NO_3-N values for RSF #3 on January 18th and 25th were an artifact caused by this RSF receiving no flow of wastewater between January 13th and January 25th. Without an input of wastewater the filter effluent recycle nitrified quite readily. With the return of wastewater flow RSF #3 still produced 12–15 mg/l of NO_3-N. In contrast the same period RSFs #2 and #4 produced 5-6 and 3-5 mg/l of NO_3-N in their effluent. Even RSF #1, which had received a hydraulic loading of

wastewater 4 times the design application rate (~0.8 m/day) since the Dec 17th restart had 1-2 mg/l of NO_3-N in the effluent.

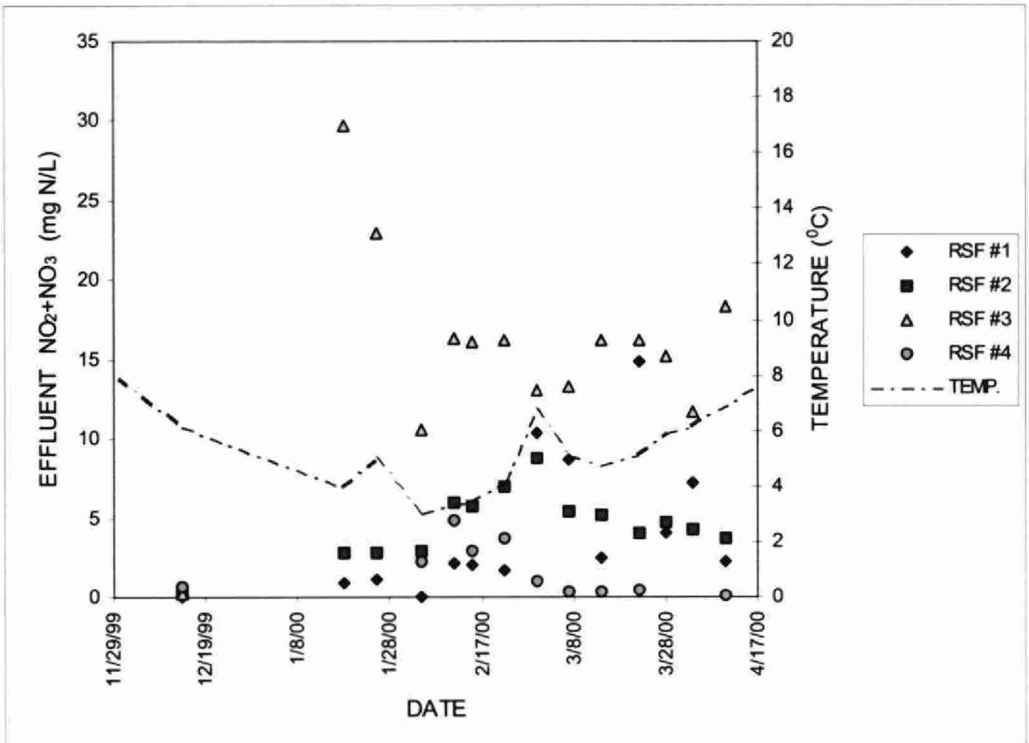

Figure 6.7: Phase 1 Nitrification

Concurrent with the development of nitrification in the RSFs during the winter period, Figure 6.8, shows that denitrification of the recycled filter effluent was occurring to a varying degree in individual RSFs. For RSF #1 the sum of TKN+NO_3-N in the RSF effluent averaged only 36 % of the influent to the second stage septic tank. This reduction strongly indicates the probability of denitrification. The effluents from RSFs #2, #3 and #4 averaged 45, 72 and 74 % respectively.

The E.Coli removals remained similar to the Acclimation Phase.

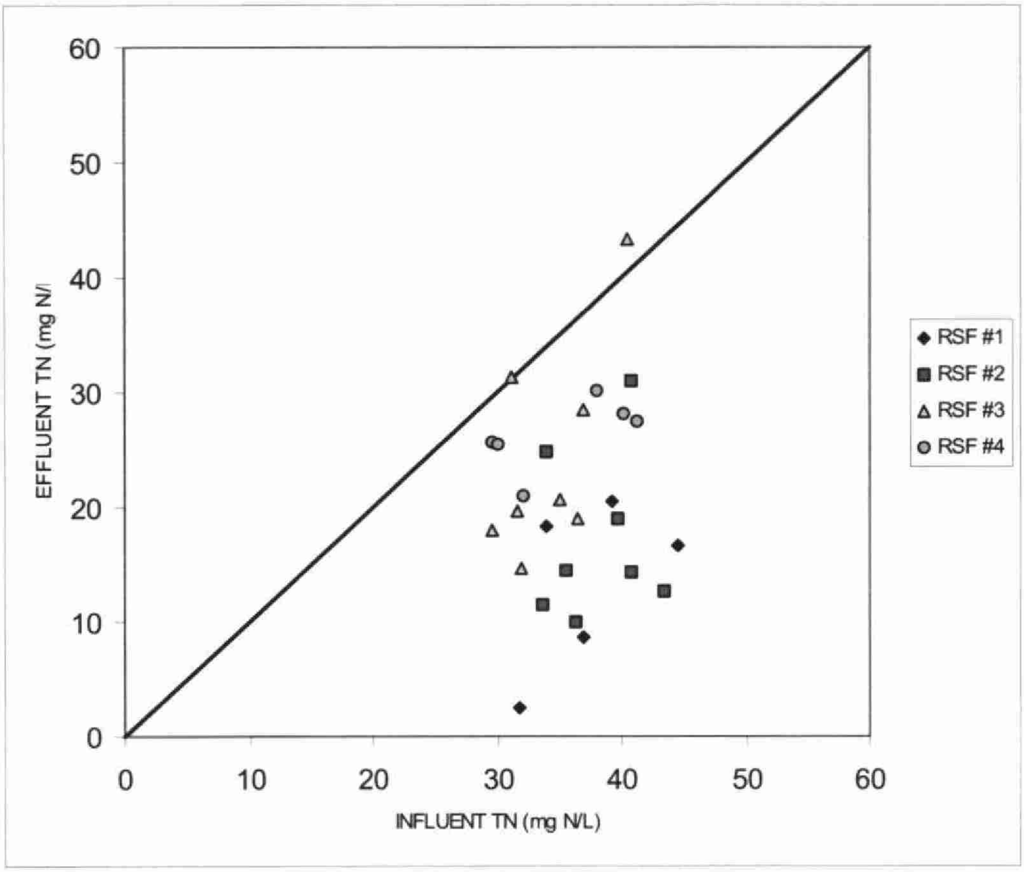

Figure 6.8: Phase 1 Nitrogen Removal

Table 6.2 Phase 1 Results (November 1999 –April, 2000)

	WASTEWATER	PRIMARY TANK EFFLUENT	RSF #1 EFFLUENT		RSF #2 EFFLUENT	RSF #3 EFFLUENT	RSF #4 EFFLUENT	
DURATION OF FLOW			12/17 – 2/25	2/28 – 4/15	11/29 – 4/15	11/29 – 4/15	12/2 – 1/16	2/6 – 3/20
MEDIA SIZE (mm)			2.6	2.6	2.6	2.6	0.16	0.16
UNIFORMITY COEFFICIENT			2	2	2	2	5	5
WASTEWATER FLOW RATE (m/d)								
DESIGN			0.2	0.2	0.2	0.2	0.20	0.20
ACTUAL			0.79	0.27	0.19	0.36	0.30	0.25
TOTAL FLOW RATE (m/d)								
DESIGN			0.6	0.6	0.6	1.0	0.60	0.60
ACTUAL			2.11	0.76	0.58	1.39	0.52	0.64
RECYCLE RATE								
DESIGN			2:1	2:1	2:1	4:1	2:1	2:1
ACTUAL			1.8:1	1.8:1	2.1:1	2.9:1	0.7:1	1.6:1
TEMP (° C)	8.5	7.1	4	4.8	5.1	5.2	5.5	4.4
D.O. (mg/L O_2)	7.3	0.8	5.6	4.9	4.6	5.6	6.2	3.6
$CBOD_5$ (mg/L)	136	93	15.8	11.5	9.3	6.7	5	19.3
TSS (mg/L)	237	61.6	17.5	17.8	15.7	14.6	19.1	14.5
NH_3+NH_4 (mg N/L)	24.4	22	16.7	6.8	8.2	3.5	23.6	25.6
NO_2 (mg N/L)	0.02	0.005	0.2	0.7	0.3	0.6	0.3	0.1
NO_3 (mg N/L)	0.07	0.09	1.2	6.6	4.4	15.0	0.4	1.9
TN (mg N/L)	44.4		25.7	17.7	18.1	23.5	27.5	27.6
E.Coli (10^3counts/100mL)	4588	7150	877	2782	386	108	0.7	360

6.3 Phase 2

The target flows to the filters were modified as shown in Table 6.3 and flow to RSFs #1, #2 and #3 commenced on May 12, 2000, and to RSF #4 on May 15, 2000. The flow for this phase to RSF #2 was erratic (detailed results presented in Appendix IV). The flow to RSF #4 was highly variable, and ponding occurred, requiring stoppage of flow on May 31st, 2000. After a reduction in target forward flow, RSF #4 was started up again on June 23rd, 2000 and shut down on July 2nd, 2000, after ponding occurred again. There was a short period of four days of forward flow just before the phase ended on August 15th, 2000. With the increase in temperature to the filters, performance improved in RSF #1, #2 and #3. The $CBOD_5$ values in the effluent were below 5 mg/L and even with the increased flows, the TSS concentrations were between 9.9 and 13.2 mg/L. Over the first part of Phase 2, RSF have an effluent $CBOD_5$ of 23.8 mg/L and an effluent TSS of 21.5mg/L. After the flow was restarted to RSF on June 22nd, the effluent $CBOD_5$ and TSS were 0.2 and 9.9 mg/L respectively. Figure 6.9 shows that all four filters were producing nitrified effluent.

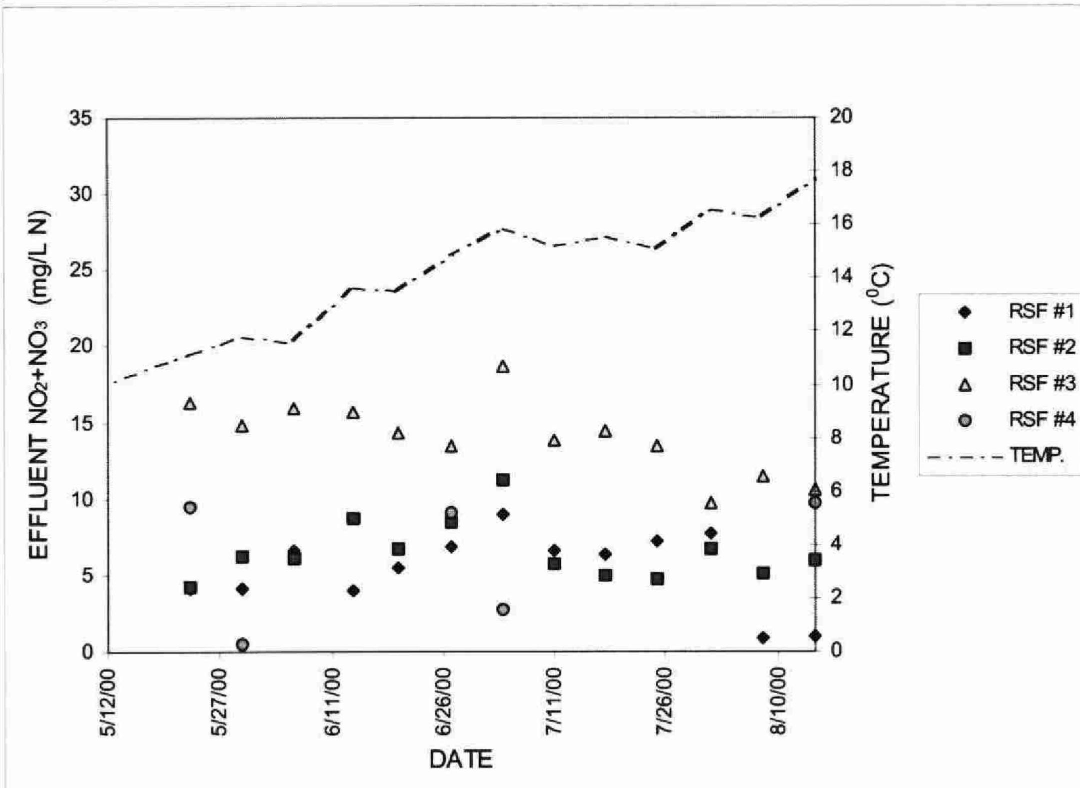

Figure 6.9: Phase 2 Nitrification

Figure 6.10 shows that nitrogen removal continued during Phase 2. For RSF #2, the effluent total nitrogen was 27.6 % of the influent to the SST. For RSFs #1 and #3, the percentage of nitrogen remaining was 32.6 and 53.2 % respectively. For RSF #4, the percentage of remaining nitrogen for the first part of the phase was 67.8 %, while for the second part of the phase it was 24.4 %. With the problems with the flow to RSF #4, there

are very few data points available, and the values quoted could vary had more data been available.
Over this phase, the E.Coli removals improved and were three orders of magnitude.

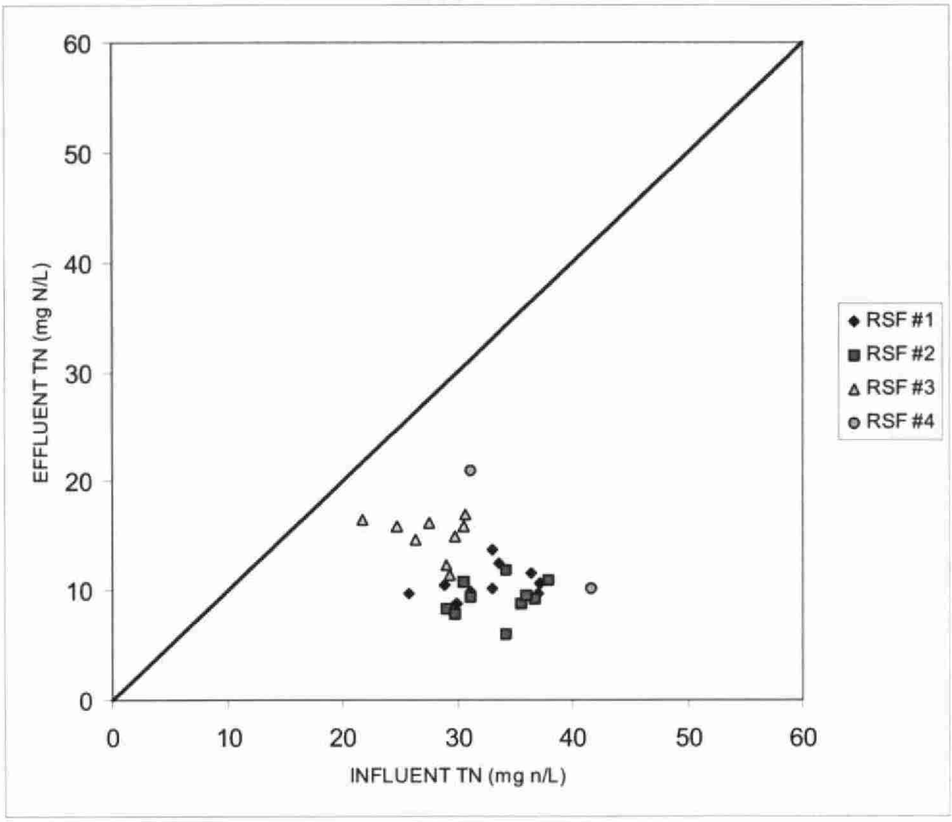

Figure 6.10: Phase 2 Nitrogen Removal

Table 6.3: Phase 2 Results (May 2000 –August 2000)

	WASTEWATER	PRIMARY TANK EFFLUENT	RSF #1 EFFLUENT	RSF #2 EFFLUENT	RSF #3 EFFLUENT	RSF #4	EFFLUENT
DURATION OF FLOW			5/12 – 8/15	5/12 – 8/15	5/12 – 8/15	5/15 –5/31	6/22 – 8/15
MEDIA SIZE (mm)			2.6	2.6	2.6	0.16	0.16
UNIFORMITY COEFFICIENT			2	2	2	5	5
WASTEWATER FLOW RATE (m/d)							
DESIGN			0.4	0.4	0.4	0.4	0.2
ACTUAL			0.39	0.4	0.39	0.28	0.23
TOTAL FLOW RATE (m/d)							
DESIGN			1.2	2.0	2.0	1.2	0.6
ACTUAL			1.07	1.77	2.0	0.72	0.52
RECYCLE RATE							
DESIGN			2:1	4:1	4:1	2:1	2:1
ACTUAL			1.7:1	3.4:1	4.3:1	1.6:1	1.3:1
TEMP (° C)	13.5	12.2	14.2	14.3	14.8	7.8	16.5
D.O. (mg/L O_2)	5	0.4	2.5	2.8	3.2	1.7	2.0
$CBOD_5$ (mg/L)	112	71	2.7	2.7	1.5	23.8	0.2
TSS (mg/L)	174	50	12.4	9.9	13.2	21.5	9.9
NH_3+NH_4 (mg N/L)	25.1	18.7	3.3	1.8	0.5	16.7	3.4
NO_2 (mg N/L)	n.d.	n.d.	0.4	0.6	0.5	0.3	0.7
NO_3 (mg N/L)	0.2	0.1	5.0	6.0	13.6	1.3	6.5
TN (mg N/L)	39.3	28.0	11.2	10.0	15.3	24.3	7.7
E.Coli (10^3counts/100mL)	12670	2960	136	19.5	3.2	777	

6.4 Phase 3

Phase 3 commenced on September 22, 2000, at which time flows were started to RSFs #1, #2 and #3. There were equipment problems with sensors on RSF #4, and flows were not started until October 30th, 2000. RSF #4 experienced further equipment problems on November 15, 2000 and although there was some flow to the filter after that date, it was deemed to be too variable for inclusion in the study. Flow details are presented in Appendix IV. For the other 3 filters, Phase 3 ended on December 20th, 2000. The TSS and $CBOD_5$ removals remained high for all four filters. Table 6.4 and Figure 6.11 show that nitrification continued during Phase 3, although at a reduced level for RSF #2, evidenced by a higher total nitrogen concentration than in Phase 2. This in part could by caused by the reduced temperatures and the increasing wastewater loading to that filter during the phase.

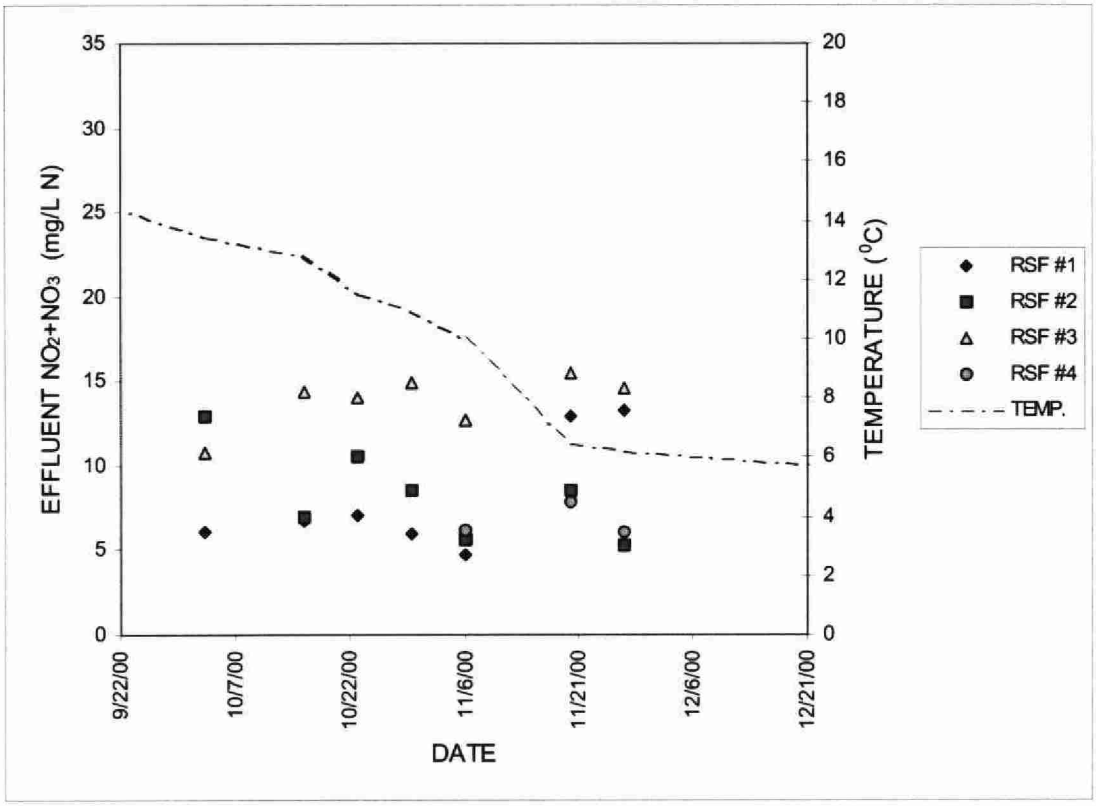

Figure 6.11: Phase 3 Nitrification

Figure 6.12 shows that nitrogen removal also continued. For RSF #1, the effluent nitrogen concentration was 28.5 % of the influent, while for RSFs #2, #3 and #4; the values were 33.8 %, 51.7 % and 39.8 % respectively.

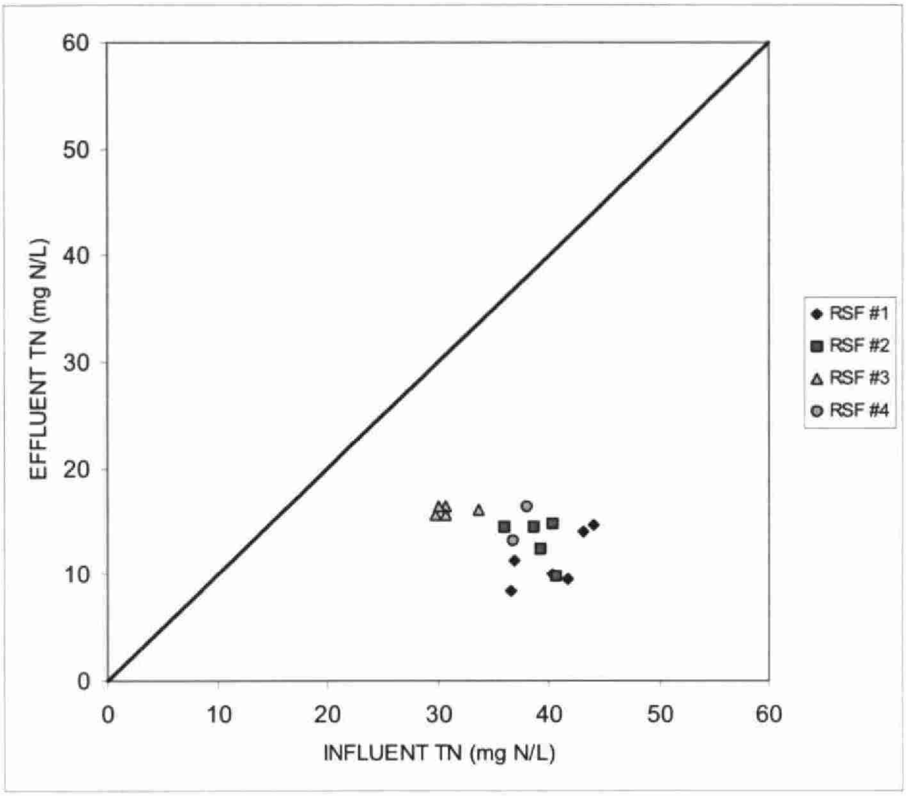

Figure 6.12: Phase 3 Nitrogen removal

Table 6.4: Phase 3 Results (September, 2000 –December, 2000)

	WASTEWATER	PRIMARY TANK EFFLUENT	RSF #1 EFFLUENT	RSF #2 EFFLUENT	RSF #3 EFFLUENT	RSF #4 EFFLUENT
DURATION OF FLOW			9/22 - 12/20	9/22 - 12/20	9/22 - 12/20	10/30 – 11/15
MEDIA SIZE (mm)			2.6	2.6	2.6	0.16
UNIFORMITY COEFFICIENT			2	2	2	5
WASTEWATER FLOW RATE (m/d)						
DESIGN			0.3	0.4	0.4	0.2
ACTUAL			0.33	0.5	0.39	0.13
TOTAL FLOW RATE (m/d)						
DESIGN			0.9	2.0	2.0	0.6
ACTUAL			0.88	1.88	2.0	0.5
RECYCLE RATE						
DESIGN			2:1	4:1	4:1	2:1
ACTUAL			1.6:1	2.8:1	4.1:1	2.9:1
TEMP (° C)	9.2	8.5	7.3	7.7	8.1	5.7
D.O. (mg/L O_2)	8.3	1.2	5.3	4.3	7	3.5
$CBOD_5$ (mg/L)	126	80	2.7	4.2	3.1	2.2
TSS (mg/L)	169	48.7	5.7	10.9	6.2	8.9
NH_3+NH_4 (mg N/L)	28.3	22.9	2.4	5.3	0.8	4.0
NO_2 (mg N/L)	n.d.	n.d.	0.7	1.0	0.4	0.6
NO_3 (mg N/L)	0.3	0.1	7.7	6.3	14.0	6.3
TN (mg N/L)	42.4	32.0	10.7	16.3	17.4	14.8
E.Coli (10^3counts/100mL)	2908	3713	11.6	33.7	66.6	0.4

6.5 Phase 4

The flow to the filters was not shut off between Phases 3 and 4 as the flow conditions were unchanged (details in Appendix IV). Due to operational problems with RSF #4, it was decided that it should not be operated during this last phase, to prevent freezing during the winter. Phase 4 was deemed to start on December 21, 2000 and continued until May 2, 2001. Heavy snow cover made sampling from some stations impossible. During this phase, wastewater and effluent samples were obtained every sampling day, but the samples from the dosing tanks were taken when conditions permitted. Table 6.5 presents the performance data for this final phase.

The performance of RSF #2 continued to deteriorate during the winter, as can be seen by the increased $CBOD_5$ and NH_3+NH_4-N average concentrations. Figure 6.13 shows the effluent nitrate for the filters for Phase 4.

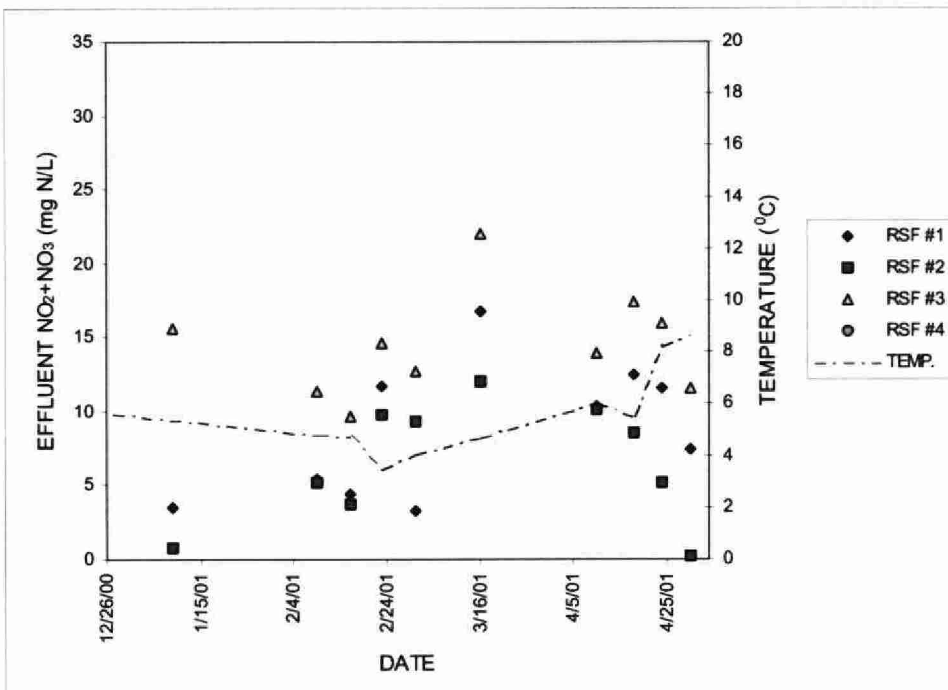

Figure 6.13: Phase 4 Nitrification

As can be seen from Figure 6.14, the nitrogen removal rate for RSF #2 also declined. The effluent concentration as a percentage of SST influent was 33.8 % in Phase 3 and increased to 52.2 % in Phase 4. For RSF #1, the rate remained very close at 29.8 %, but there was some reduction for RSF #3, from Phase 3's 51.7 % to 61.2% in Phase 4.

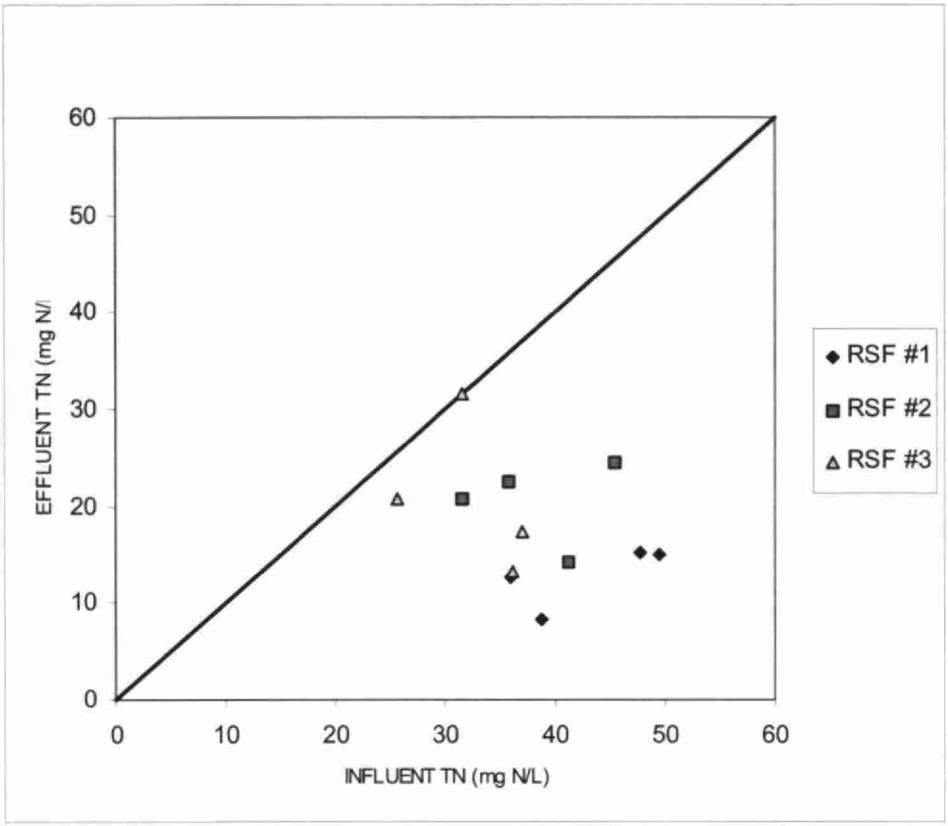

Figure 6.14: Phase 4 Nitrogen Removal

Table 6.5: Phase 4 Results (December, 2000 – May, 2001)

	WASTEWATER	PRIMARY TANK EFFLUENT	RSF #1 EFFLUENT	RSF #2 EFFLUENT	RSF #3 EFFLUENT
DURATION OF FLOW			12/21-5/02	12/21-5/02	12/21-5/02
MEDIA SIZE (mm)			2.6	2.6	2.6
UNIFORMITY COEFFICIENT			2	2	2
WASTEWATER FLOW RATE (m/d)					
DESIGN			0.3	0.4	0.4
ACTUAL			0.34	0.45	0.37
TOTAL FLOW RATE (m/d)					
DESIGN			0.9	2.0	2.0
ACTUAL			0.84	1.89	2.03
RECYCLE RATE					
DESIGN			2:1	4:1	4:1
ACTUAL			1.5:1	3.2:1	4.5:1
TEMP (° C)	7	6	5.1	5.5	5.9
D.O. (mg/L O_2)	12.5	1.9	7.5	5.6	9.6
$CBOD_5$ (mg/L)	130	83	2.9	6	4.8
TSS (mg/L)	137	43.8	4.8	9.7	6.2
NH_3+NH_4 (mg N/L)	22.3	24.1	2.8	13.3	0.8
NO_2 (mg N/L)	n.d.	n.d.	0.7	1.0	0.1
NO_3 (mg N/L)	0.3	0.2	7.9	5.6	14.4
TN (mg N/L)	27.4	31.8	10.6	19.8	19.5
E.Coli (10^3counts/100mL)	2763	3425	4.7	50.3	102

6.6 Sludge Accumulation

The amount of sludge accumulated in the septic tanks will determine how often these tanks have to be pumped. Sludge depth in the PST and SSTs was measured after one year of operation and again at the end of the study (further 9 months of operation). When the measurements were performed, it was noted that in some instances there was a thick layer and then usually a loosely settled layer on top. The data presented in Table 14 shows these results. It is interesting to note that at the end of the study, SST 1 and SST 2 had no sludge in them. This could possibly be attributed to the recycling of the effluent back to this chamber. For RSF #3, the recycle did not go back to SST 3, but to the dosing tank. Consequently, sludge was still present in the tank. It is also interesting to note that at the end of the study, the level of sludge in the PST was the same as after one year of operation. For this study, no means of phosphorus removal was investigated. Depending on location, addition of chemical for P removal would probably increase the sludge accumulation rate.

Table 6.6 Sludge Accumulation

	August 15, 2000		April 30, 2001	
	Total Sludge (cm)	Thick Sludge (cm)	Total Sludge (cm)	Thick Sludge (cm)
PST	35.6	7.5	35.6	7.5
SST 1	33	15	0	0
SST 2	7.5	7.5	0	0
SST 3	20	7.5	15	0
SST 4	33	20	n/a	n/a

6.7 Filter Loading

In surveying the literature, one of the criteria used for filter failure was plugging. Generally, plugging indicated that the filter had developed enough biological material so that the influent could no longer readily pass through and be treated. This could lead to anaerobic conditions and deteriorating effluent quality. Once a filter had 'plugged', then total loading could be established. With exception of RSF #4, none of the other 3 filters showed signs of 'plugging' after the geo-textile fabric was removed from the filters. RSFs #1, #2 and #3 operated from September 22, 2000 to May 2, 2001 (223 days) without any evidence of this type of failure. As a consequence, rather than evaluate total loading, daily mass loading rates for each filter were determined. From these rates, maximum loading rates that would still give effluent within the required could be determined.

With the high variability in daily flows, and the occasional missing flow numbers (as discussed in Appendix IV), it was decided that a seven day running average of flow values would be used for loading calculations. Loading rates were determined for $CBOD_5$, TSS, NH_3+NH_4 and TKN, and the results are summarized below in Table 6.7.

Table 6.7: Average Daily Mass Loading Rates

	$CBOD_5$ LOADING	TSS LOADING	AMMONIA+ AMMONIUM LOADING	TKN LOADING
	($g/m^2/d$)	($g/m^2/d$)	($g/m^2/d$)	($g/m^2/d$)
ACCLIMATION PHASE (Sept 1999 - Nov 1999)				
RSF #1	14.0	9.8	14.49	22.27
RSF #2	23.6	15.6	24.02	29.56
RSF #3	25.8	18.7	26.70	32.34
RSF #4	27.3	16.2	20.17	24.11
PHASE 1 (Nov 1999 -Apr 2000)				
RSF #1	15.9	16.7	7.33	10.36
RSF #2	13.4	18.5	9.32	13.44
RSF #3	27.5	30.1	17.05	25.63
RSF #4	22.9	18.4	12.82	12.40
PHASE 2 (May 2000 - Aug 2000)				
RSF #1	21.3	25.3	9.83	15.68
RSF #2	18.7	27.1	9.19	16.01
RSF #3	10.8	30.2	6.67	12.46
RSF #4	13.6	14.9	7.97	11.22
PHASE 3 (Sept 2000 - Dec 2000)				
RSF #1	15.3	16.8	6.80	10.53
RSF #2	12.1	37.6	11.82	22.12
RSF #3	15.3	83.1	10.61	17.68
RSF #4	8.8	9.8	5.24	8.26
PHASE 4 (Dec 2000 - May 2001)				
RSF #1	13.0	11.4	4.45	8.19
RSF #2	33.2	31.2	12.36	15.21
RSF #3	42.3	82.1	16.96	25.41

From all the daily loading data available, slopes were determined for each filter and each phase. Table 6.8 presents these slopes. They can be used to predict maximum loading rates for a required effluent quality. More detailed results are available in Appendix V.

Table 6.8: Slope for Mass Removal vs. Effluent Concentration

	$CBOD_5$	TSS	AMMONIA + AMMONIUM	TN	TN vs Eff NO_x
	($g/m^2/d$ (influent) vs mg/L(effluent))				
ACCLIMATION PHASE (Sept 1999 - Nov 1999)					
RSF #1	0.45	0.88	1.28	1.10	0.01
RSF #2	0.71	0.39	1.01	1.19	0.00
RSF #3	0.53	0.30	0.91	0.90	0.00
RSF #4	0.84	0.36	1.07	1.21	0.00
PHASE 1 (Nov 1999 - Apr 2000)					
RSF #1	0.60	0.97	1.06	1.58	0.72
RSF #2	0.60	0.93	1.48	1.52	0.16
RSF #3	0.26	0.50	0.59	0.69	0.21
RSF #4	0.43	0.68	1.32	1.63	0.07
PHASE 2 (May 2000 - Aug 2000)					
RSF #1	0.10	0.44	0.28	0.61	0.30
RSF #2	0.13	0.33	0.17	0.42	0.27
RSF #3	0.14	0.31	0.06	0.47	0.43
RSF #4	0.17	0.42	0.42	1.02	0.39
PHASE 3 (Sept 2000 - Dec 2000)					
RSF #1	0.19	0.42	0.33	0.86	0.63
RSF #2	0.20	0.39	0.27	0.33	0.22
RSF #3	0.11	0.01	0.09	0.42	0.38
RSF #4	0.26	1.31	0.58	1.17	0.76
PHASE 4 (Dec 2000 - May 2001)					
RSF #1	0.26	0.39	0.66	1.17	0.94
RSF #2	0.20	0.25	0.27	0.65	0.31
RSF #3	0.08	0.02	0.02	0.38	0.34

7.0 DISCUSSION OF RESULTS

The pilot facility at the Clifford STP suffered its share of delays and initial operational problems. Unfortunately not all of these were resolved. Operation of the pilot filters, over the first winter of operation indicated that the use of a fine media at the selected hydraulic loading rate, coupled with an hourly dosing frequency, caused surface ponding, which in cold weather resulted in surface freezing. The fine media filter was not operated during the second winter. On the other hand, the effluent quality during the first winter, for the coarse media pilot RSFs, was consistently equivalent to typical municipal biological treatment plants. This effluent quality for $CBOD_5$ and TSS was maintained throughout the study, consistently below 5 mg/L for CBOD and 10 mg/L for TSS through phase 2, 3 and 4. The only excursion outside these values was for RSF #2. During Phase 3, the TSS was 10.9 mg/L and during Phase 4, the $CBOD_5$ was 6 mg/L.

Despite filtered effluent temperatures, which remained in the 3-6^{0}C range, the RSFs started to nitrify, producing a partially nitrified effluent. In addition denitrification was observed. Under these cold weather conditions, a discharge effluent with 5-7 mg/l of total ammonia nitrogen and 4-6 mg/l of nitrate nitrogen has been produced from a raw wastewater averaging 29 mg/l of total ammonia nitrogen and 50 mg/l of total nitrogen.

During Phase 2, nitrification and denitrification increased with resulting effluent nitrate + nitrite values for RSF #1 and #2 of 12.4 and 9.9 mg N/L. The corresponding ammonia + ammonium values were 3.3 and 1.8 mg N/L. RSF#3, which had less possibility for denitrification because it recycled to the dosing tank, had effluent values of 14.1 mg N/L for the nitrate + nitrite and 0.5 mg N/L for ammonia + ammonium. RSF #4 was operated for two brief periods during Phase 2 and achieved nitrification over the latter part of the phase (6.5 mg N/L of nitrate + nitrite and 3.4 mg N/L of ammonia + ammonium).

For Phase 3, the flow to RSF #1 was reduced, while flows to the remaining filters were maintained. With the decreasing temperatures, the rate of both denitrification and nitrification was reduced. Effluent nitrate + nitrite values for RSF #1, #2 and #3 were 8.4, 7.3, and 14.4 mg N/L respectively. The corresponding ammonia + ammonium values were 2.4, 5.3 and 0.8 mg N/L. RSF #4 operated for a very short time, but did appear to nitrify and denitrify.

Phase 4 flows remained the same as Phase 3, but the effluent quality deteriorated further. This could be a result of the lower temperatures. The temperatures averaged 5.1 °C with a minimum of 2.5 °C for RSF #1. For RSF #2, the corresponding values were 5.5 °C and 3 °C, while for RSF #3 they were 5.9 °C and 4.8 °C respectively.

8.0 CONCLUSIONS

The experimental design was formulated to focus on the following design and operating conditions of recirculating intermittent sand filters. In some instances, the variability in flow and consequent variability in recycle ratio precluded any conclusions.
The effect of hydraulic wastewater loadings (up to 0.6 m/day) substantially higher than present industry standards (0.2 m/day) on filter performance. The increased loading caused a reduction in effluent quality.
The relative importance of wastewater concentrations or loadings (i.e. $CBOD_5$, TSS and Total Ammonia Nitrogen) applied to the filter. The high variability in flows prevented prediction of maximum loading rates with any certainty.
The effect on filter performance of mixing different quantities of recycled effluent to the wastewater prior to application to the filter. The addition of recycled effluent adds to the hydraulic loading but decreases the applied concentrations that may affect filter effluent concentrations. The high variability in all flows precluded recommendations.
The effect on filter performance and costs by the variation of the time between applications of wastewater to the filters, and to determine the preferable frequency of application. Hourly dosing produced higher quality effluent that twice-daily effluent. Due to flow variability, it was difficult to determine if more frequent dosing (3 times per hour) produced even higher quality effluent that hourly dosing.
The advantages and disadvantages of a finer media, as used in existing Ontario intermittent sand filters, in comparison to the coarser media currently used in many recirculating intermittent sand filters.
The requirements to maximize nitrification and denitrification to provide an effluent low in both total ammonia and nitrate nitrogen
A comparison of the performance of a conventional RSF to an MOE approved commercial (Orenco) RSF.

These results indicate that wastewater may be treated year round under Ontario environmental conditions using a septic tank followed by a recirculating intermittent sand filter using the coarser media.

9.0 RECOMMENDATIONS

From the conclusions above, it can be seen that there are areas needing additional work. In order to optimize the system, a better means of measuring flow is required. In-line flow meters should be installed, and the pumps for application to the filter should be replaced. Currently, these pumps are over-sized, and have led to a number of operational problems. The control system should be simplified, and there should be more regular monitoring of the pilot facility, either on-site or by remote signal. It would be beneficial to continue operation over two cold weather periods, i.e. for two winters.

In addition, phosphorus removal should be investigated and optimized both from a removals standpoint, and from a sludge accumulation standpoint

10.0 FUTURE WORK

There are several areas that should be looked at in further detail. Phosphorus removal should be investigated, both in terms of achieving the required levels for Ontario and also its impact on sludge accumulation and frequency of required sludge removal.

Different media types and/or sizes should be investigated, or possibly a mixed media size i.e. a slightly finer media with the coarse media on top. The mixed media filter would improve effluent solids removal and increase the quality of the effluent. A number of different types of media have been suggested, e.g. slag, cullet.

The media depth for this pilot study was 0.610 m, whereas most ISFs and RSFs have media depths of 0.9-1.0. The possibility of increasing media depth should be considered. The possibility of even coarser gravel on top to minimize plant growth should also be considered. Under present conditions, plant growth readily occurs on the surface of all the filters. This could lead to problems with filter performance unless the filters are regularly maintained. Figure 9.1 shows the growth within six weeks of filter maintenance. Alternately, one filter could be covered with a removable cover to reduce growth. This cover might also improve filter operation by reducing the impact of cold weather conditions. With a cover, it is very important that adequate ventilation is provided to the filter surface and underdrains to prevent any possibility of anaerobic conditions. Covering is probably only feasible for smaller installations, and a surface maintenance schedule for larger filters should be established.

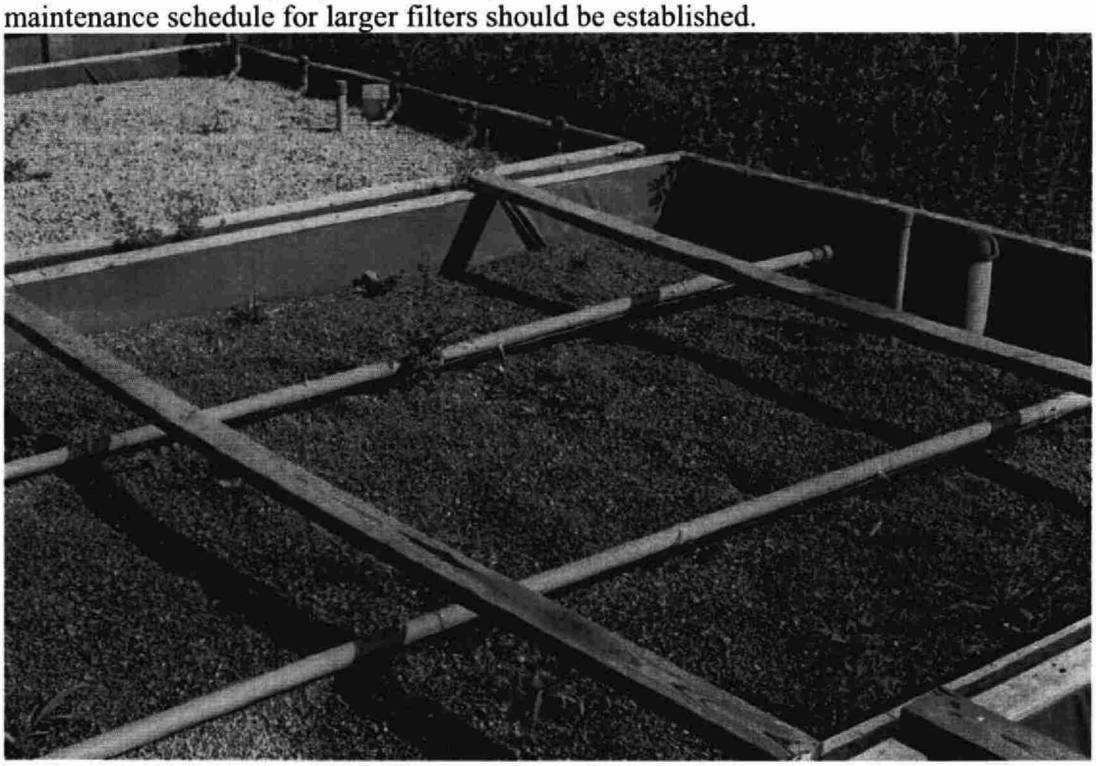

Figure 10.1: RSFs #2 and #3 Showing Plant Growth

During this study, the longest time that the filters were operated without a rest was 223 days. A filter should be set up and left to operate under ideal conditions so that a better idea of filter run length can be determined. The three coarser filters did not show hydraulic failure after 223 days and RSF #1, which operated close to ideal conditions, continued to produce effluent of very high quality.

If it is possible to achieve better flow measurement and control for the pilot facility, than a closer investigation of the impact of recycle ratios and hydraulic loadings could be performed.

Various means of effluent disinfection should be evaluated. The low levels of effluent TSS would indicate a good potential for use of UV lights for disinfection. Without any disinfection, E. Coli removals of 3 orders of magnitude were achieved during Phases 3 and 4 for RSF #1, #2 and #3 and 4 orders of magnitude for RSF #4 during Phase 3.

11.0 BIBLIOGRAPHY

Environmental Protection Agency Design Manual "Onsite Wastewater Treatment and Disposal Systems", EPA 625/1-80-012, October, 1980.

McMaster University, "Application of Intermittent Sand Filters to Improve Effluent Quality", Interim Report for Ontario Ministry of the Environment and Great Lakes 2000 Cleanup Fund, K. L. Murphy & L. A. Robertson, 1998

R.V. Anderson Associates Ltd., XCG Consultants Ltd., Alternative Approaches for Upgrading Effluent Quality for Lagoon Based Systems, Ontario Ministry of the Environment and environment Canada report, 1992

APPENDICES

APPENDIX I: CONTROL SYSTEM

The experimental system is controlled by a custom commercial control panel supplied by Orenco. The digital control (Figure AI.1) consists of a microprocessor and the associated circuitry to allow communication over a standard telephone line, digital outputs for the control of relays or other on/off events, digital inputs to sense switch closures or other Orenco inputs, and finally, analog inputs to sense the analog outputs from pressure transducers or other analog devices.

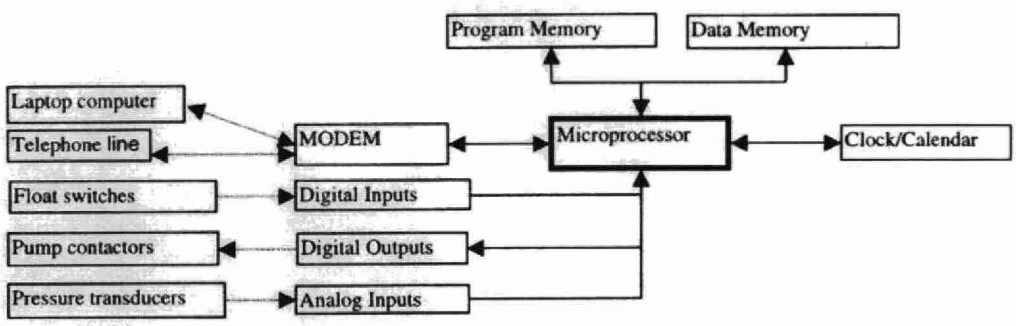

Figure AI.1: Schematic of Process Control

A Clock/Calendar within the circuitry supplies the time and date and a user specified program sequence controls the timing of events and data collection. Batteries on the circuit board retain memory and the clock/calendar in case of power outage. A serial output is available to connect a local computer, or terminal, for on site program maintenance or data download.

Programming is accomplished either by a local computer, operated in terminal mode, or remotely over the modem. All programming done by the end user is accomplished through a series of menus wherein the user selects the macro to be performed and enters the arguments for the macro.

During the initial phases of the experiment, and due to the complex nature of the experimental system, several new macros were written and downloaded to the controller board. Examples of macros include:

turning pump on or off if pressure sensor reading equals or exceeds a limit,
turning pump on or off if a switch on/off condition is met,
calculate the total run time/day of a pump,
calculate the total on/off events/day,
transmit the stored data to the remote computer, and
establishing timing cycles for pumps.

The remaining control panel box contains the high current contactors for actuating the pumps, a set of manual override switches to allow any pumps to be manually switched on or off, and the connection block to allow wiring of the external wires by an electrician without disturbing the internal interconnections.

Access from a remote location allows the user to inspect the status of all controlled functions as well as to change the control macros and their arguments. A remote

computer could be programmed to dial the control panel, download data, check the status, and if necessary, execute emergency measures in case of a problem.
Numerous changes to the basic programming were needed to accommodate the nature of the experimental system and the operating environment. For instance, power outages are frequent as testing of the emergency generators is a routine maintenance procedure for the sewage treatment plant. Many of the original voltage based pressure transducers failed within a few weeks and were replaced with current based ones.

APPENDIX II

AII.1 Chemical Oxygen Demand

Chemical oxygen demand (COD) tests were performed on unfiltered samples. The presence of particulates can lead to large variations from one test to another. McMaster analyses were performed using Standard Methods (1992) method 5220D and a Hach DL/2000 programmable spectrophotometer, while MOE analyses used MOE method E3246A.

As shown in figure AII.1, the MOE values were generally lower than those obtained at McMaster.

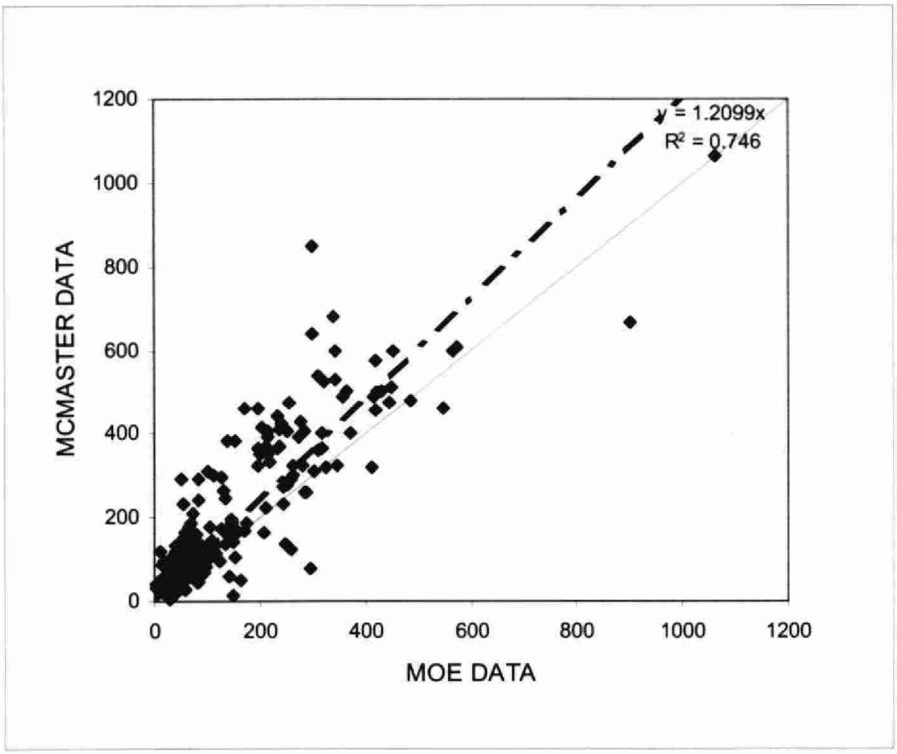

Figure AII.1: Chemical Oxygen Demand

AII.2 Total Suspended Solids

These analyses were performed on total samples, with the presence of particulates leading to greater variability. Analyses at McMaster were performed following Standard Methods (1992) method 2540D, while those at MOE used MOE procedure SS3188. At McMaster, sample sizes of 250 mL were used for all but the raw sewage and effluent from the primary tank. A sample size of 50 mL was used for raw sewage and 100 mL for the primary tank effluent. A Mettler HL 52 balance, with accuracy to 0.01mg was used. Again, as shown in figure AII.2, the McMaster values were generally higher than those obtained by MOE.

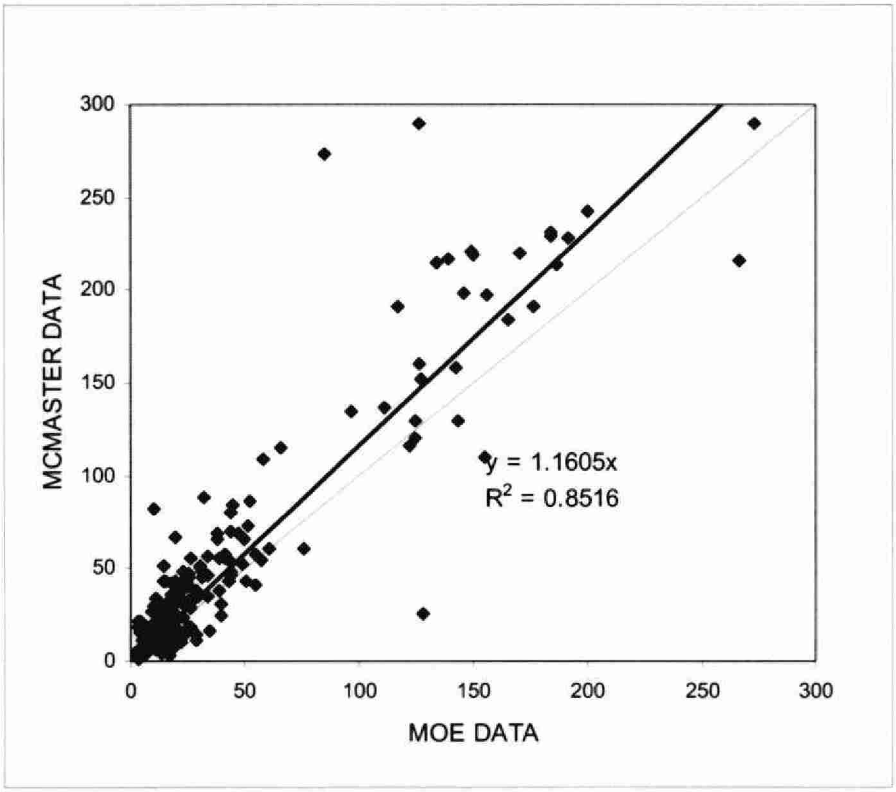

Figure AII.2: Total Suspended Solids

AII.3 Ammonia

At McMaster, ammonia tests were performed on filtered samples using the Hach Nessler method. The maximum concentration using this method is 2.5 mg/L as N and samples were diluted as required to fall within this value. MOE uses method E3366A. As can be seen in the accompanying figure, the MOE values are slightly higher than the McMaster ones but generally there is good agreement.

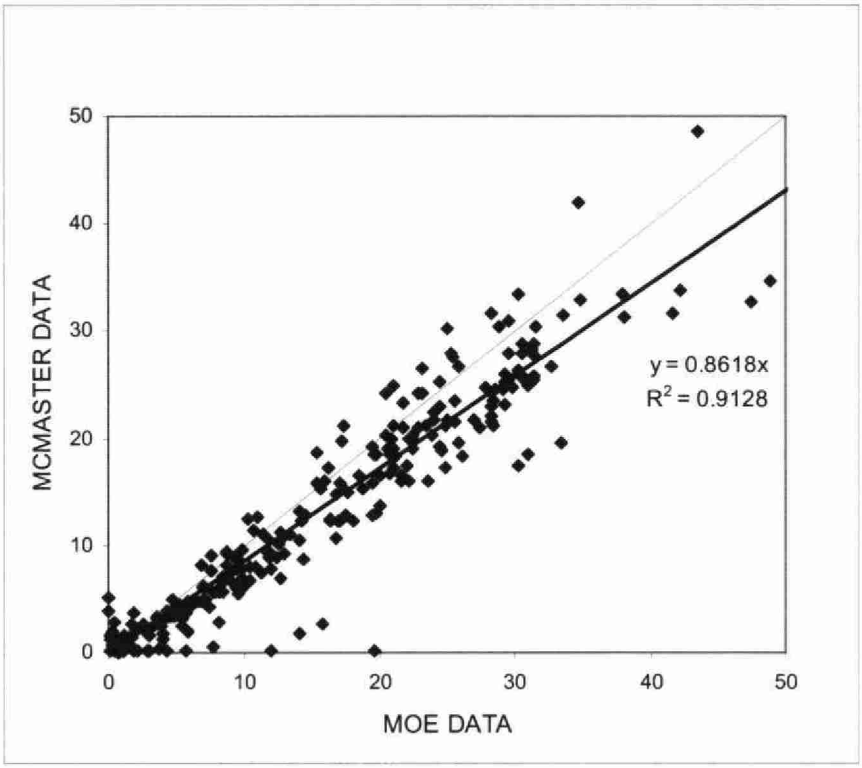

Figure AII.3 Ammonia + Ammonium

AII.4 Chloride

Chloride analyses at McMaster were performed using a Dionex 4500I on filtered samples. (see Standard Methods (1992) method 4110A). The chloride concentrations were very high, affecting the ionic strength of the sample and hence the concentration. Samples for chloride were diluted by a factor of 10 to eliminate this problem. Chloride at MOE was measured using MOE method E3016A.

There was significant scatter in the data, but the mean values were in close agreement, as can be seen in the figure.

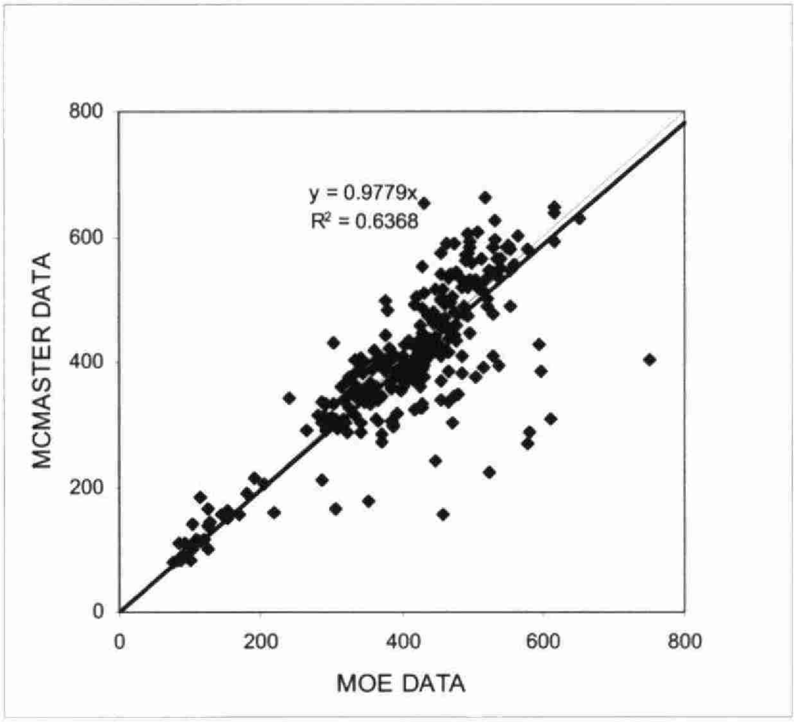

Figure AII.4: Chloride

AII.5: Nitrate

Nitrate analyses at McMaster were performed on filtered samples using the Dionex 4500I ion chromatograph. (see Standard Methods (1992) method 4110A). MOE analyses were performed using MOE method E3366A.

There was close agreement between the two laboratories, as can be seen in the figure.

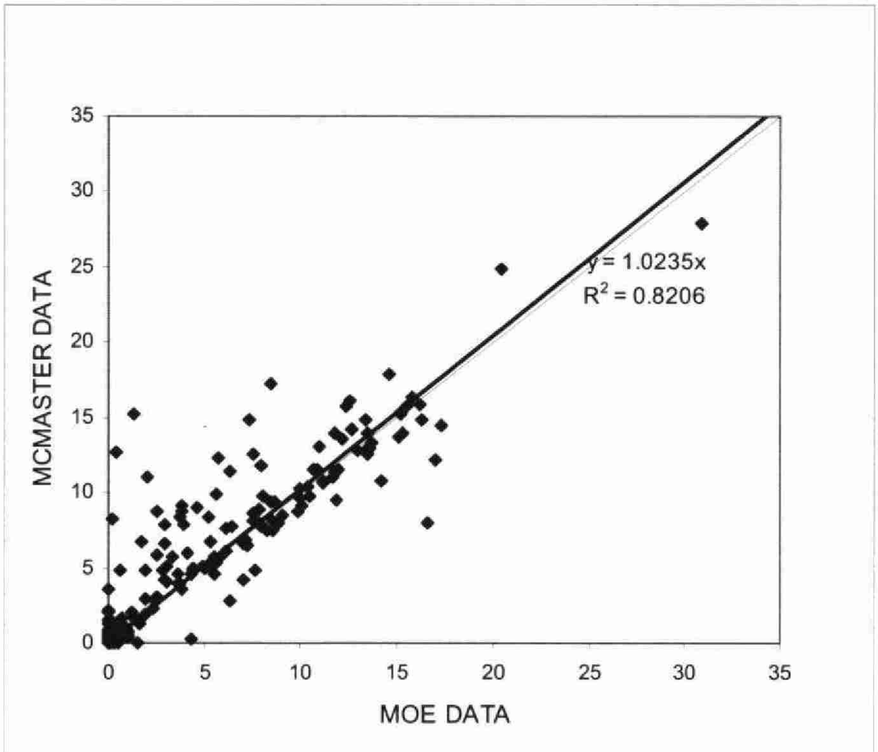

Figure AII.5: Nitrate

AII.6 Phosphate

Phosphate analyses at McMaster were performed on filtered samples using the Dionex 4500I ion chromatograph. (see Standard Methods (1992) method 4110A). At MOE, the method used was E3366A.

As can be seen on accompanying figure, the trendline on the graph is fairly close to the 1:1 line, but there is substantial scatter in the values. Phosphate is a weak acid ion and its dissociation can vary depending on a number of factors (pH, ionic strength, alkalinity). This could explain the scatter in the results.

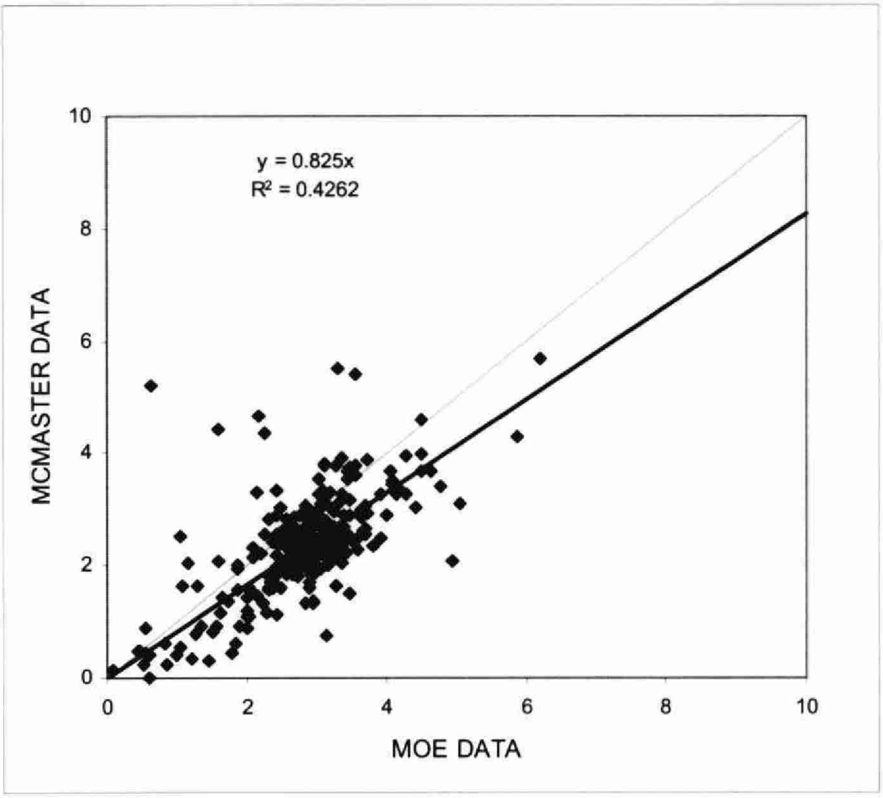

Figure AII.6: Phosphate

APPENDIX III: SAND ANALYSIS

AIII.1 Fine Sand

The fine sand was that was specified in the design was meant to be equivalent to that used at Clifford and at other sand filters in existence in Ontario. A sieve analysis was performed on the sand as installed in RSF#4 and is presented in Figure AIII.1. This sand had a D10 of 0.1 mm and a uniformity coefficient (UC) of 2.4. The sieve analysis for this sand can be seen in Figure AII.1. During the Acclimation Phase, significant hydraulic problems were encountered, and it was decided to replace it with slightly coarser sand. This sand was replaced with the sand shown in Figure AIII.2, having a D10 of 0.16 mm and a UC of 5.

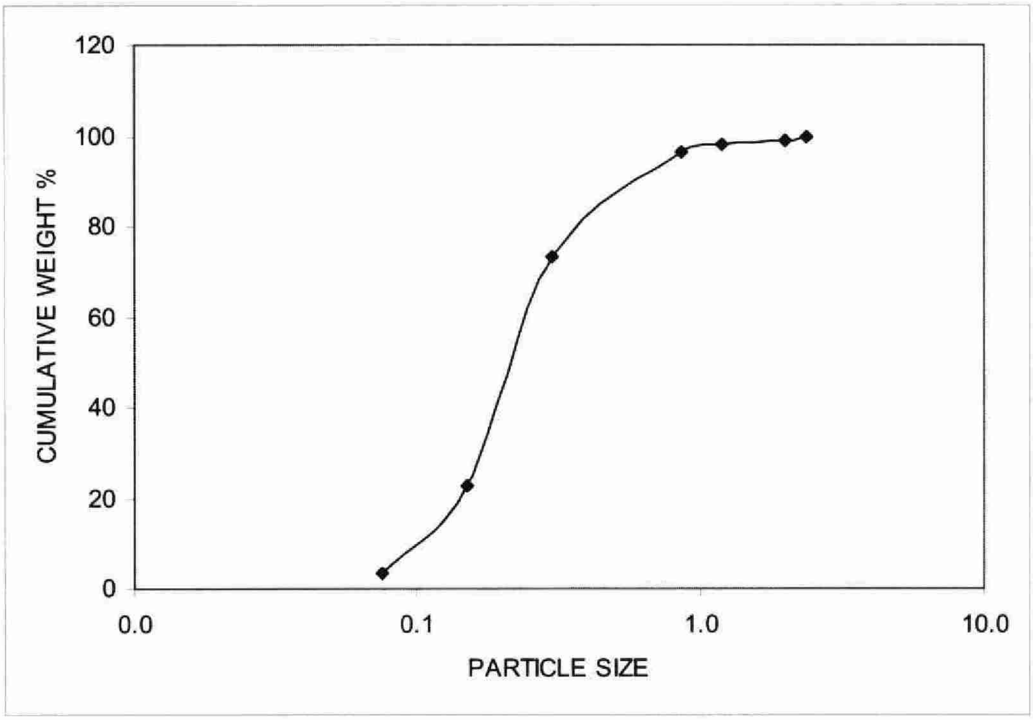

Figure AIII.1: Fine Sand Used in Acclimation Phase

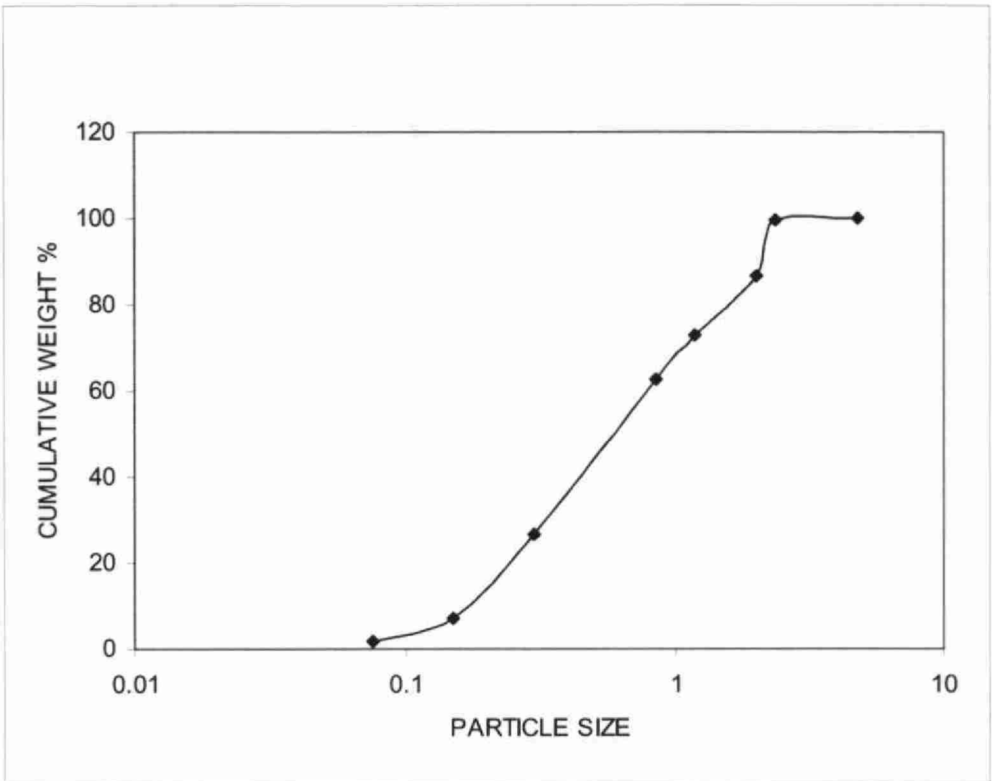

Figure AIII.2: Fine Sand Used in Phases 1, 2 and 3

AIII.2 Coarse Sand

The coarse sand as installed was used through the entire pilot study in RSFs #1, #2 and #3. The D10 was 2.6 mm and the UC was 2. Figure AIII.3 shows the sieve analysis for this sand.

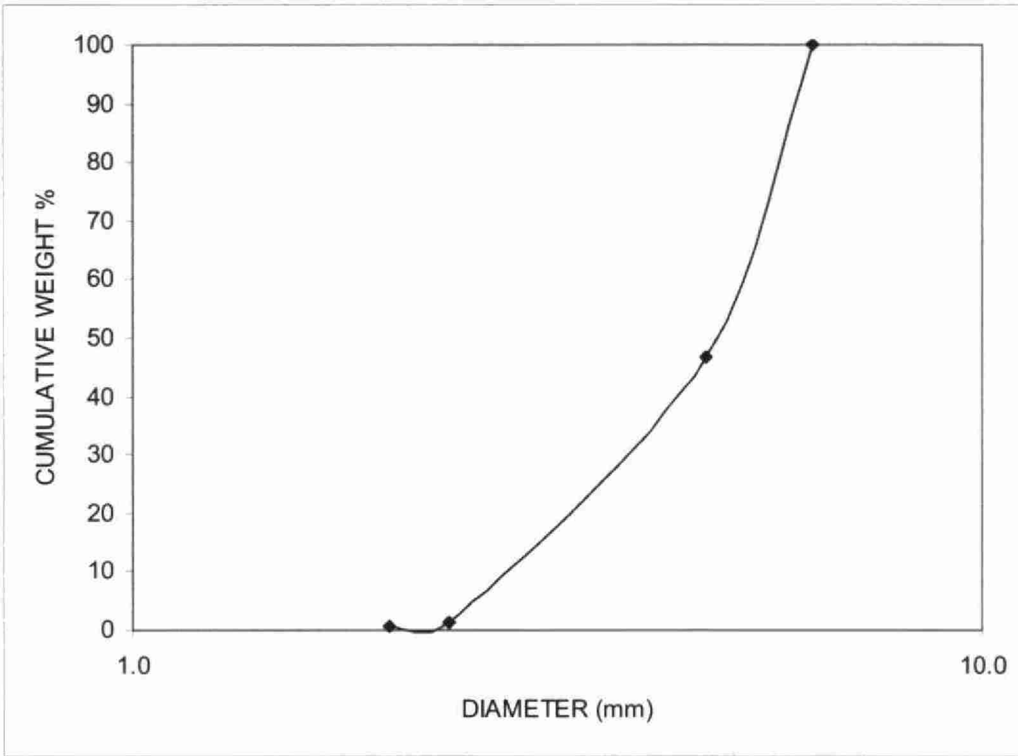

Figure AIII.3: Coarse Sand Used in Entire Study for RSFs #1, #2, #3.

APPENDIX IV: DETAILED FLOW INFORMATION

AIV.1 RSF #1

RSF #1 was constructed with coarse media and had the capability for either hourly or daily dosing. The design and actual hydraulic loadings to RSF #1 for the entire study (September 1999 to May 2001) are summarized in Table AIV.1. All values used for experimental flows and recycle ratios are mean values.

Table AIV.1: RSF#1 Flow Summary

Phase		Acc'n	1		2	3	4
Date		9/13-11/1	12/17-2/25	2/29-4/15	5/12 – 8/15	9/22 – 12/20	12/21 – 5/02
Wastewater (m/d)	Design	0.20	0.20		0.4	0.3	0.3
	Mean	0.23	0.79	0.27	0.39	0.33	0.34
	Std. Dev.	0.08	0.17	0.2	0.07	0.05	0.10
	Coef. of Var.	0.35	0.21	0.74	0.18	0.14	0.29
Total (m/d)	Design	1.00	0.60		1.2	0.9	0.9
	Mean	0.61	2.11	0.76	1.07	0.88	0.84
	Std. Dev.	0.31	0.6	0.56	0.21	0.01	0.12
	Coef. of Var.	0.51	0.28	0.74	0.20	0.02	0.14
Recycle Ratio	Design	4:1	2:1		2:1	2:1	2:1
	Mean	2.5:1	1.8:1	1.8:1	1.7:1	1.6:1	1.5:1
Dosing Frequency		1/hr	2/24 hours		1/hr	1/hr	1/hr

During the Acclimation Phase, the wastewater flow to RSF #1 from Sep. 13th to Oct. 8th and from Oct. 25th to Nov 2nd was variable (Coef. of Var. 0.35) ranging from 0.13 to 0.37 m/day if the days with no wastewater flow (October 9, 1999 to Oct 24, 1999) are not included. The mean wastewater flow for both these periods, 0.23 m/day, approximated the experimental design objective of 0.20 m/day. The actual total flow to the filter (0.61 m/day) was even more variable, caused by variability in the amount recycled from the splitter. The Coef. of Var. was 0.51. The recycle never attained the design total flow of 1.0 m/day. The actual recycle ratio for both periods was 2.5:1 or 64 % of the design value.

During October, the performance of RSF #1 began to deteriorate. The effluent had shown atypically low effluent Dissolved Oxygen (DO) concentrations ranging between 1 and 3 mg/l. By early November RSF #1 showed a tendency to pond and high values of biochemical oxygen demand ($CBOD_5$ between 26 and 33 mg/l) began to appear in the effluent. Field investigation showed that geotextile cloth placed over the RSFs underdrains had become clogged and was restricting the flow through the RSFs. This was causing anoxic conditions. Wastewater flow to the RSFs #1, was stopped on November 1st. The geotextile cloth was replaced with coarse gravel media. Wastewater flow was resumed to RSF #1 on December 17th, this was deemed to be the start of Phase 1. Figure AIV.1 shows the Acclimation Phase flows.

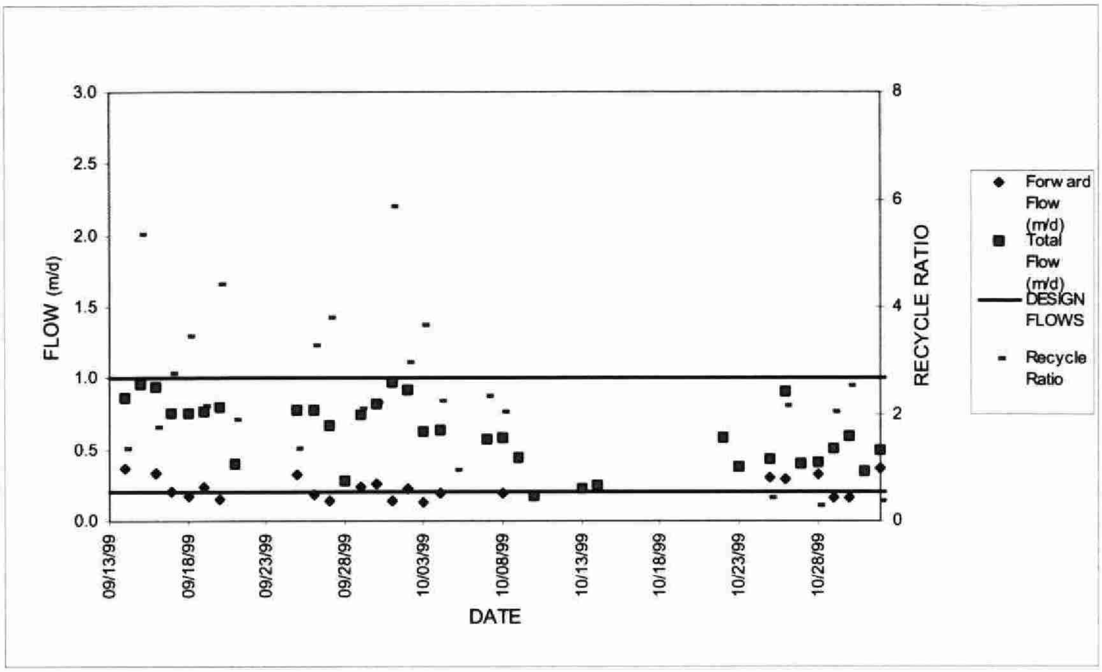

Figure AIV.1: RSF #1 Acclimation Phase Flows

Prior to restarting the pilot plant, the flow to RSF #1 was set to the design values of 0.2 m/d forward flow and 0.6 m/d total flow with a resulting recycle ratio of 2:1 for the winter period as shown in Table 4. The dosing frequency was modified from hourly to daily.

Following start up for the winter period, Fig. AIV.2 indicates the wastewater flows to RSF #1 were highly variable ranging from 0.08 to 1.12 m/day from Dec 17th to Feb 25th. In addition the mean wastewater flow of 0.79 m/day was 395 % of the design flow of 0.20 m/day. Similarly, the total applied flow ranged from 0.3 to 2.56 m/day with a mean of 2.11 m/day or 352 % of the design flow of 0.60 m/day. The actual recycle ratio was 1.77:1, lower than the design value. The change over of RSF #1 from the hourly dosing tank to the daily dosing tank caused a problem with the control program, which in turn activated the wastewater pump and resulted in these high wastewater flows. A new control program was initiated on February 12th and a faulty pressure transducer was replaced on February 28th. Wastewater flows for the remainder of the winter period (until April 15th) were still highly variable (Coef. of Variation of 0.74) and mean 0.27 m/day or 135 % of design. The applied total flow had similar variability (Coef. of Variation of 0.74) and a mean flow of 0.76 m/d or 127 % of the design flow. It would appear that the splitter tended to provide a relatively constant recycle flow and did not respond to the flow variations in the applied wastewater flow. The actual recycle ratio for this period was 1.8:1.

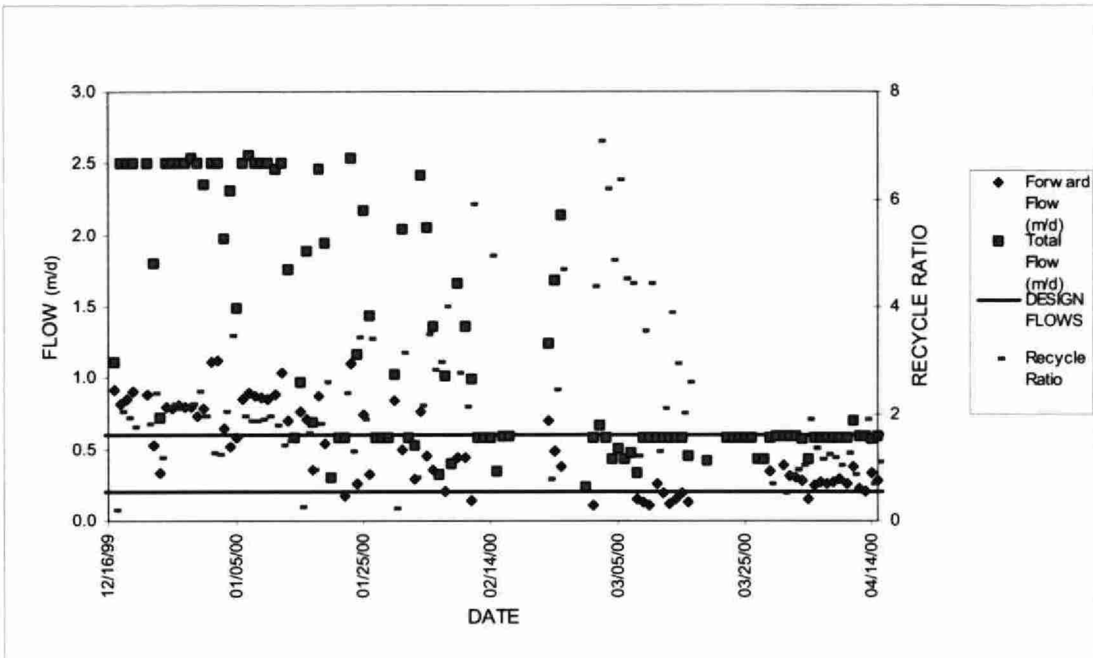

Figure AIV.2: RSF #1 Phase 1 Flows

During the break after Phase 1, the flows were set to the design values of 0.4 m/d wastewater flow and 1.2 m/d total flow. The dosing was returned to hourly. The actual wastewater flow to RSF #1 was erratic at the start of Phase 2, but eventually settled down to very close to the design flow until July 28th. At this time, problems were reported with both the flow splitter and with a stone being caught in the pump intake pipe. The total flow followed the same pattern. The mean flows were 0.33 m/d for wastewater and 1.07 m/d for total flow, with a recycle ratio 1.7. The line from the main wet well to the primary tank became blocked at the beginning of August and flows to all four RSFs became erratic from this time. Phase 2 was terminated on August 15th.

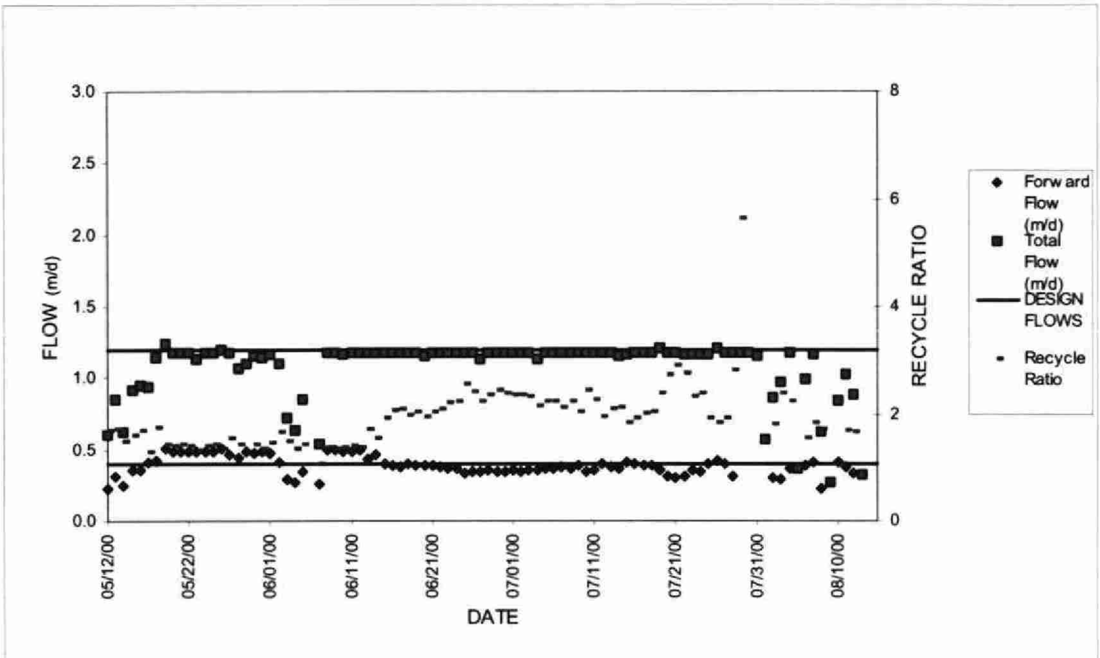

Figure AIV.3: RSF #1 Phase 2 Flows

During August 2000, attempts were made to correct the problem with the pump in the main wet well. It was determined the line was partially blocked, and to clear this blockage the pump would have to be removed. Due to manpower and safety constraints, this would not have been possible for some time. Since the flow balances (Table 4.3) indicated relative agreement between the main pump and the individual pumps, it was decided to continue with the next phase.

For Phase 3, the design flows were reduced to 0.3 m/d wastewater flow and 0.9 m/d total flow, with a resultant recycle ratio of 2:1. Phase 3 commenced on September 22nd. During Phase 3, the mean wastewater flow was 0.34 m/d and the total flow was 0.88 m/d. The forward flow was slightly over the design, (113 %) and the total slightly under (98%). Consequently, the recycle ratio was 1.6:1 as opposed to the design value of 2:1.

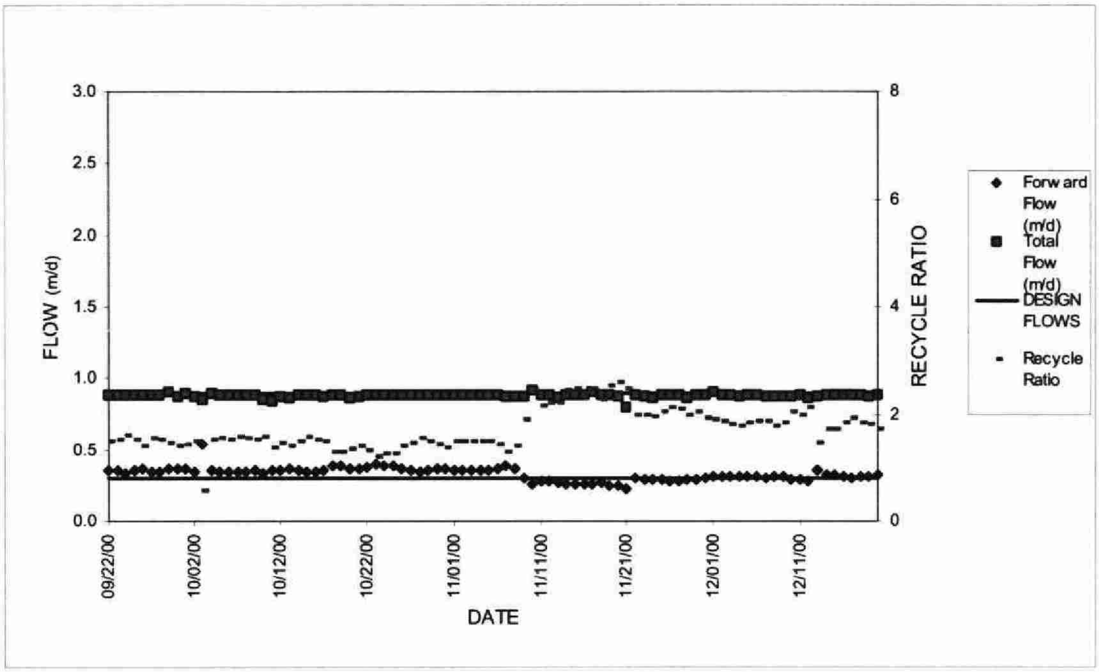

Figure AIV.4: RSF #1 Phase 3 Flows

The final phase commenced on December 21st, with no changes in the settings for flow. Phase 4 ended May 2nd 2001, at which time the pilot facility was shut down for maintenance and calibration. For two short periods in late February and early March, the wastewater flow had to be turned off as the municipality was disposing of excess sludge.

The wastewater flow to RSF #1 at 0.34 m/d, was 113 % of design and showed quite a degree of variability (minimum flow 0.13 m/d to maximum flow 0.73 m/d), whereas the total flow (0.84 m/d) was 93 % of design with less variability (minimum 0.27 m/d to maximum 0.91 m/d). The recycle ratio was 1.5, which is 75% of design. These values were close to those achieved in Phase 3.

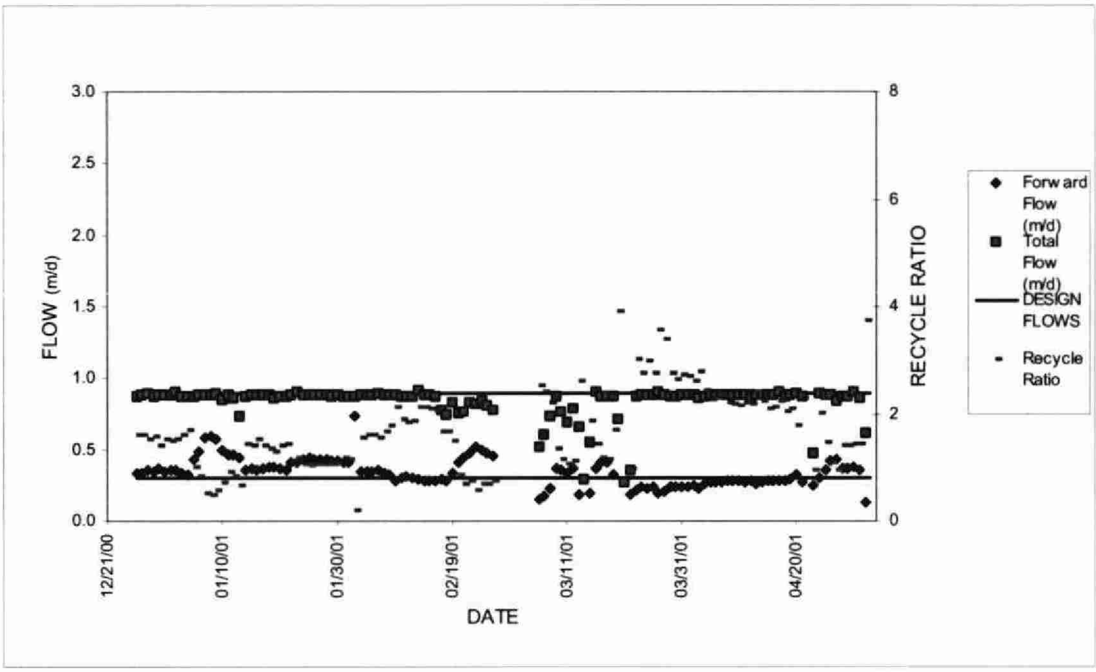

Figure AIV.5: RSF #1 Phase 4 Flows

AIV.2 RSF #2

RSF #2 had the same coarse media as RSF #1, but only had hourly or more frequent dosing capability. Table AIV.2 shows the design flow, actual flow and recycle values for RSF #2 for this pilot study.

Table AIV.2: RSF#2 Flow Summary

Phase		Acc'n	1	2	3	4
Date		9/13-11/1	11/29-4/15	5/12 – 8/15	9/22 – 12/20	12/21 – 5/02
Wastewater (m/d)	Design	0.20	0.20	0.4	0.4	0.4
	Mean	0.24	0.19	0.40	0.50	0.45
	Std. Dev.	0.06	0.02	0.1	0.11	0.17
	Coef. of Var.	0.25	0.11	0.25	0.22	0.38
Total (m/d)	Design	1.00	0.60	2.0	2.0	2.0
	Mean	0.88	0.58	1.77	1.88	1.89
	Std. Dev.	0.24	0.06	0.51	0.24	0.18
	Coef. of Var.	0.27	0.10	0.29	0.13	0.09
Recycle Ratio	Design	4:1	2:1	4:1	4:1	4:1
	Mean	2.6:1	2.1:1	3.4:1	2.8:1	3.2:1
Dosing Frequency		1/hr	1/hr	1/hr	3/hr	3/hr

The wastewater flow to RSF #2 during the Acclimation Phase had a variability ranging from 0.12 to 0.39 m/day with a Coefficient of Variation of 0.25. The mean wastewater flow (0.24 m/day) was 20 % in excess of the design value of 0.20 m/day. The relatively consistent recycle flow from the splitter meant that the actual total flow, with a mean value of 0.88 was equally variable (Coef. of Var. 0.27). However, the actual recycle ratio was only 2.6:1 or 65 % of the design value of 4.0. As with RSF #1, RSF #2 showed deterioration in effluent quality during October and November. The geotextile cloth used was replaced with gravel before Phase 1 commenced on November 29th.

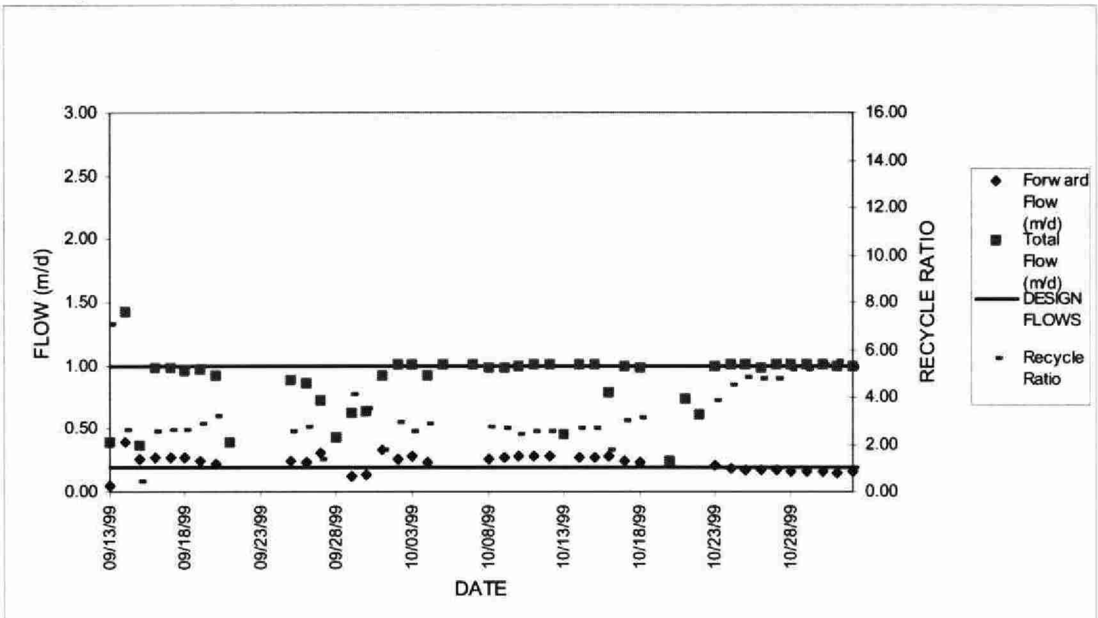

Figure AIV.6: RSF #2 Acclimation Flows

During Phase 1, with the exception of the short period when one day of zero wastewater flow was applied to the RSF on January 14, 2000, both wastewater and total applied flows to RSF #2 were consistent and approximated the experimental design values. Excluding this exception, the range of actual wastewater flows was from 0.11 to 0.24 m/day and the actual total flow from 0.12 to 0.65 m/day. This would indicate consistent performance of the feed pump and the recycle splitter. The wastewater flow of 0.19 m/day was 95 % of the design value and the total applied flow of 0.58 m/day was 97 % of the design flow. The recycle ratio was 2.1:1 or 105 % of design.

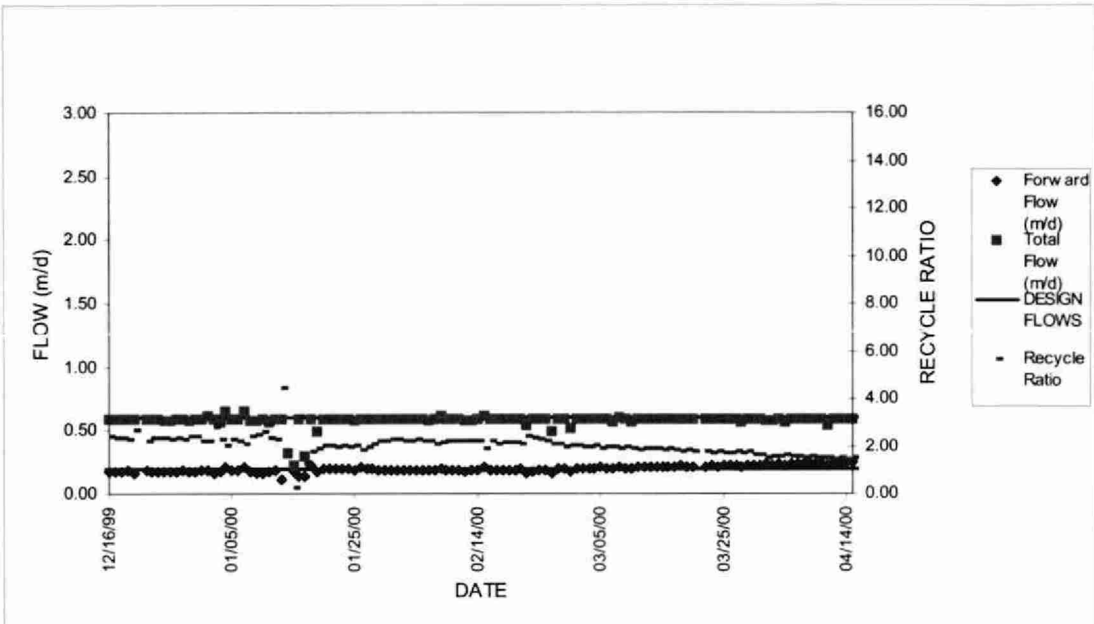

Figure AIV.7: RSF #2 Phase 1 Flows

For Phase 2, the design wastewater flow was increased to 0.40 m/d and the total flow to 2.0 m/d, with a target recycle ratio of 4:1. The wastewater flow was 0.40 m/d and the total flow was 1.77 m/d (88.5 % of design). The recycle ratio was 3.4:1 as opposed to the target of 4:1. There was substantial variability in both flows, the wastewater flow varied from 0.10 to 0.59 m/d and the total flow from 0.15 to 2.82 m/d.

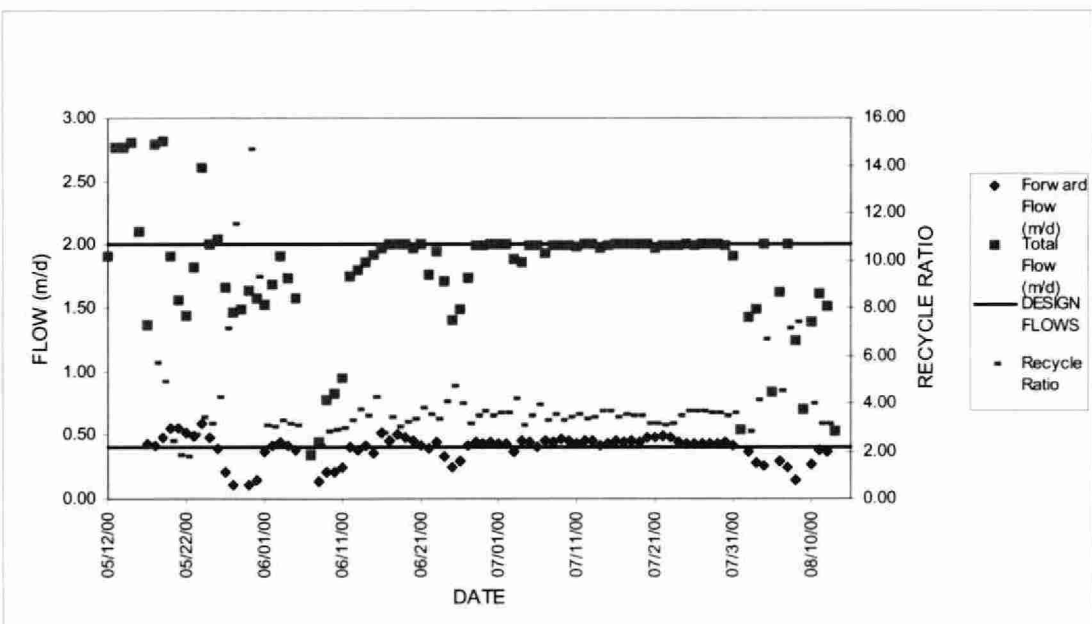

Figure AIV.8: RSF #2 Phase 2 Flows

The design values for flow remained the same for Phase 3, but the application frequency to RSF #2 was changed from hourly to 3 times each hour. Both wastewater and total flows to RSF #2 were very erratic at the commencement of this phase but stabilized on

October 15th. There was no indication in the data received from SFI if modifications to equipment had been made to achieve this stability. The total flow was very close to design for the rest of the phase, but the forward flow stabilized at 0.5 m/d and gradually increased to 0.7 m/d by the end of the phase. The mean wastewater and total flows were 050 m/d and 1.88 m/d respectively. This resulted in a recycle rate of 2.8:1, substantially below the target of 4:1. Over the phase, the wastewater flow varied from 0.10 to 0.61 m/d. The total flow varied from 0.79 to 2.03 m/d.

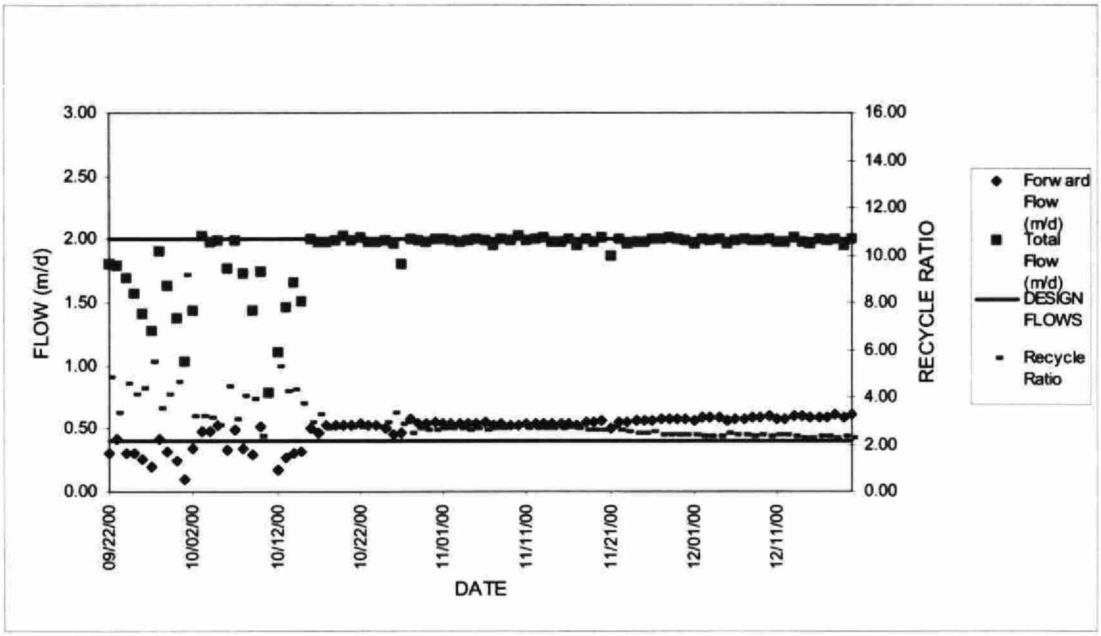

Figure AIV.9: RSF #2 Phase 3 Flows

For Phase 4, all design setting remained the same. The mean values for this phase were wastewater 0.45 m/d, total 1.89 m/d, and recycle 3.2:1. Again, there was substantial variability at the beginning and end of the phase due to problems with equipment settings and equipment malfunction. As a consequence of this variability, the Coef. Of Var. for wastewater flow was 0.38. The total flow was less variable, and the Coef . of var. was 0.09.

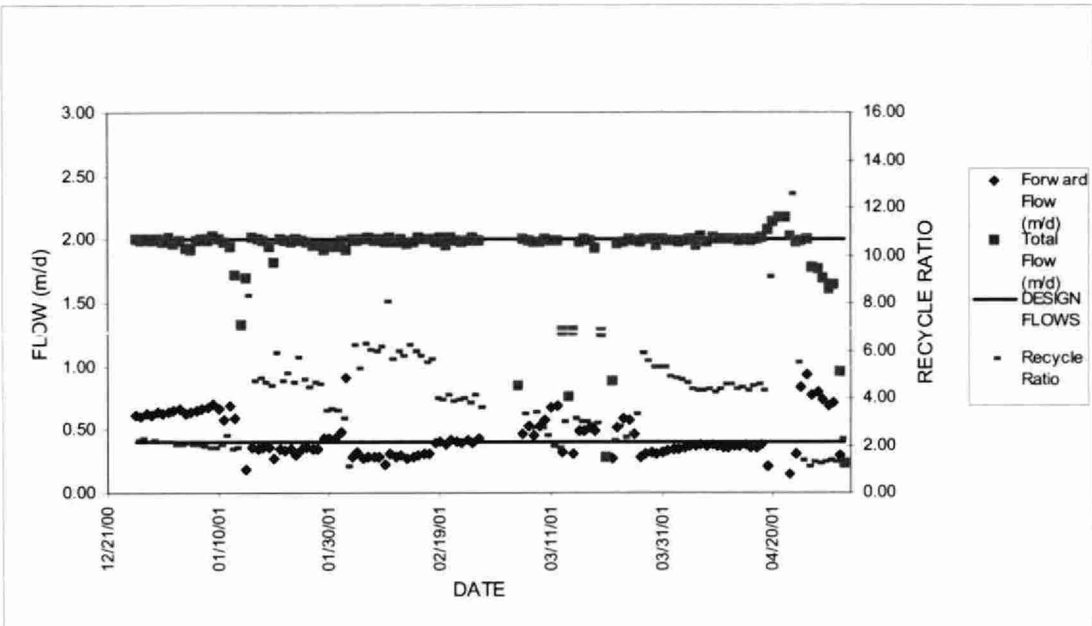

Figure AIV.10: RSF #2 Phase 4 Flows

AIV.3 RSF #3

RSF #3 was the ORENCO filter. The media was equivalent to RSFs #1 and #2, but dosing frequency, method of dosing and recycle point were different. The design and actual flow values are presented in Table AIV.3.

Table AIV.3: RSF #3 Flow Summary

Phase		Acc'n	1	2	3	4
Date		9/13-11/11	11/29-4/15	5/12 – 8/15	9/22 – 12/20	12/21 – 5/02
Wastewater (m/d)	Design	0.20	0.20	0.4	0.4	0.4
	Mean	0.34	0.36	0.35	0.39	0.37
	Std. Dev.	0.08	0.11	0.08	0.09	0.14
	Coef. of Var.	0.24	0.31	0.23	0.22	0.38
Total (m/d)	Design	1.00	1.00	2.0	2.0	2.0
	Mean	0.94	1.39	1.84	2.0	2.03
	Std. Dev.	0.10	0.51	0.28	0.03	0.40
	Coef. of Var.	0.11	0.37	0.55	0.07	0.2
Recycle Ratio	Design	4:1	4:1	4:1	4:1	4:1
	Mean	1.8:1	2.9:1	4.3:1	4.1:1	4.5:1
Dosing Frequency		3/hr	3/hr	3/hr	4/hr	4/hr

From Figure AIV.11 it can be seen that the actual wastewater flow to RSF #3 was relatively consistent for the Acclimation Phase. The Coef. of Variation was 0.24. The flow ranged from 0.13 to 0.44 m/day. Unfortunately the median wastewater flow for the period (0.34 m/day) exceeded the design flow by 70 %. A consistent recycle flow ensured that actual total flow was also consistent, averaging 0.94 m/day – this despite on occasion

ranging from 0.41 to 1.47 m/day. The low Coefficient of Variation was 0.11. The actual recycle ratio was only1.8:1 or 45 % of the design value. Again, as with RSFs #1 and #2, deteriorating effluent quality necessitated the replacement of the geotextile cloth before Phase 1 could start on November 29th.

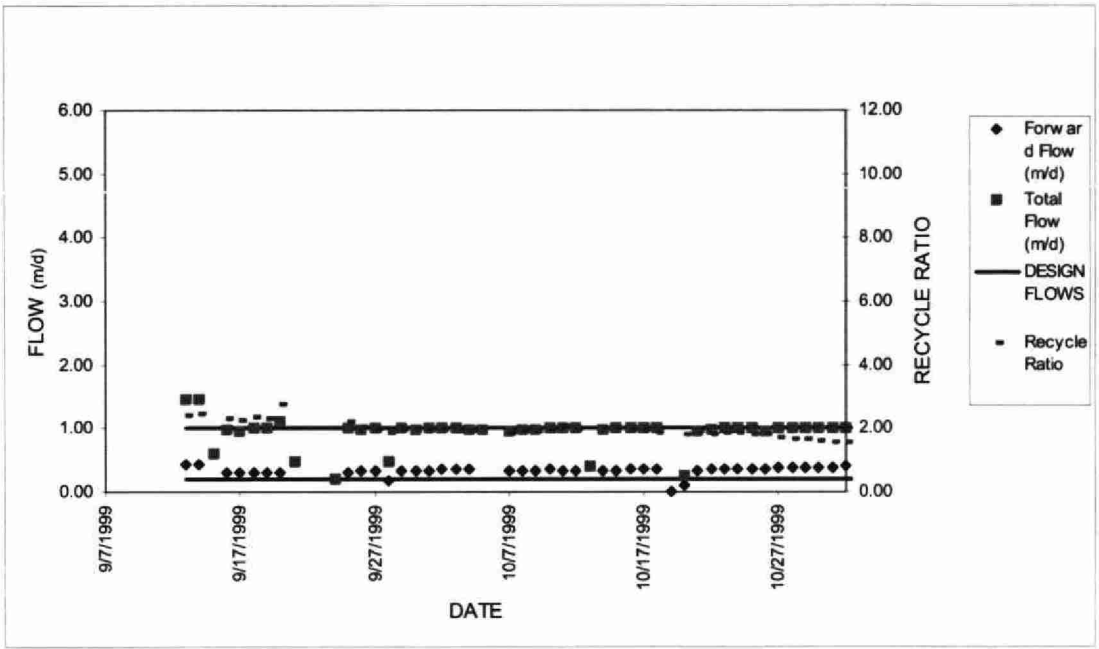

Figure AIV.11: RSF #3 Acclimation Flows

The design settings remained the same for Phase 1 as they were in the Acclimation Phase. The actual wastewater flow to RSF #3 was relatively variable ranging from 0.16 to 0.69 m/day and averaged 180 % higher (0.36 m/day) than the design value of 0.2 m/d. The total applied flow was approximately 139 % of the design value. This would give a recycle ratio of approximately 2.9:1 or 72.5 % of the design value of 4:1. The total applied flow was considerably more variable ranging from 0.06 to 3.73 m/day. Severe flow excursions of both wastewater and total flows occurred in January (Jan 16 to 24) and in total applied flow in late February and early March (Feb 27 to Mar 8) and early April (Mar 30 to Apr 15), possibly caused by faulty pressure transducers.

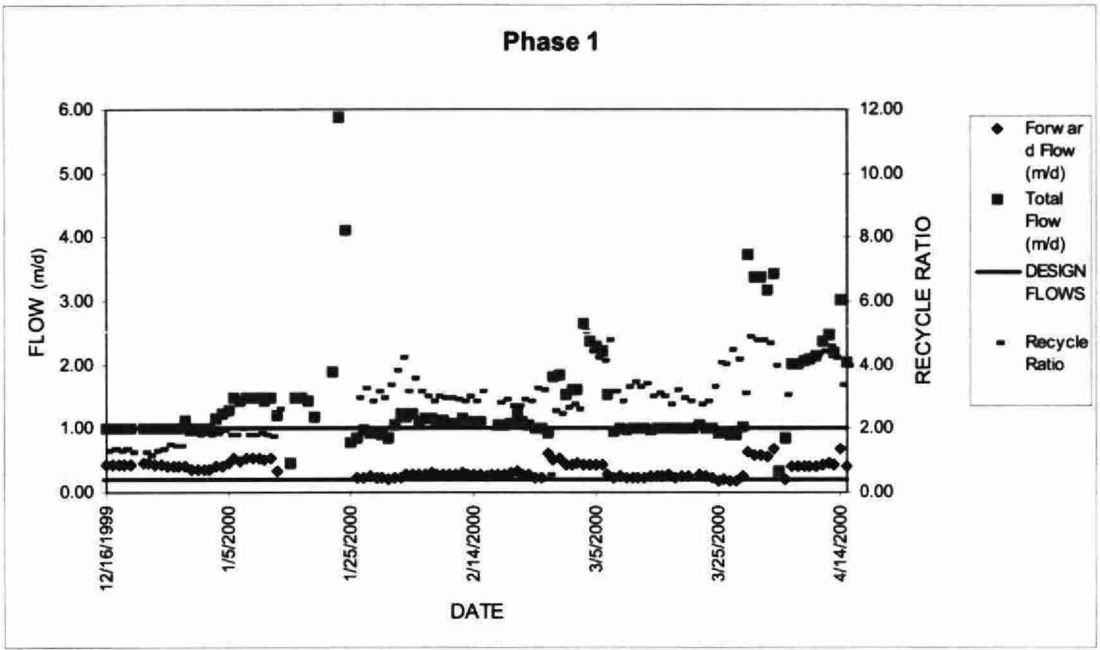

Figure AIV.12: RSF #3 Phase 1 Flows

For Phase 2, the design flows were adjusted to 0.4 m/d wastewater and 2.0 m/d total flow, and the dosing frequency was increased from 3 to 4 times per hour. During this phase, the actual flows for RSF #3 were less variable than those for RSF #1 and #2 and were very close to design. The forward flow was 88 % of design while the total flow was 92% of design. The resulting recycle ratio was 4.3:1, slightly higher than the design. The flows to RSF #3 were very stable. The wastewater flow started at just under 0.5 m/d and gradually decreased to 0.2 m/d. Although the average recycle ratio was very close to design for the phase, the actual recycle ratio increased from 3.3 at the beginning to almost 8.

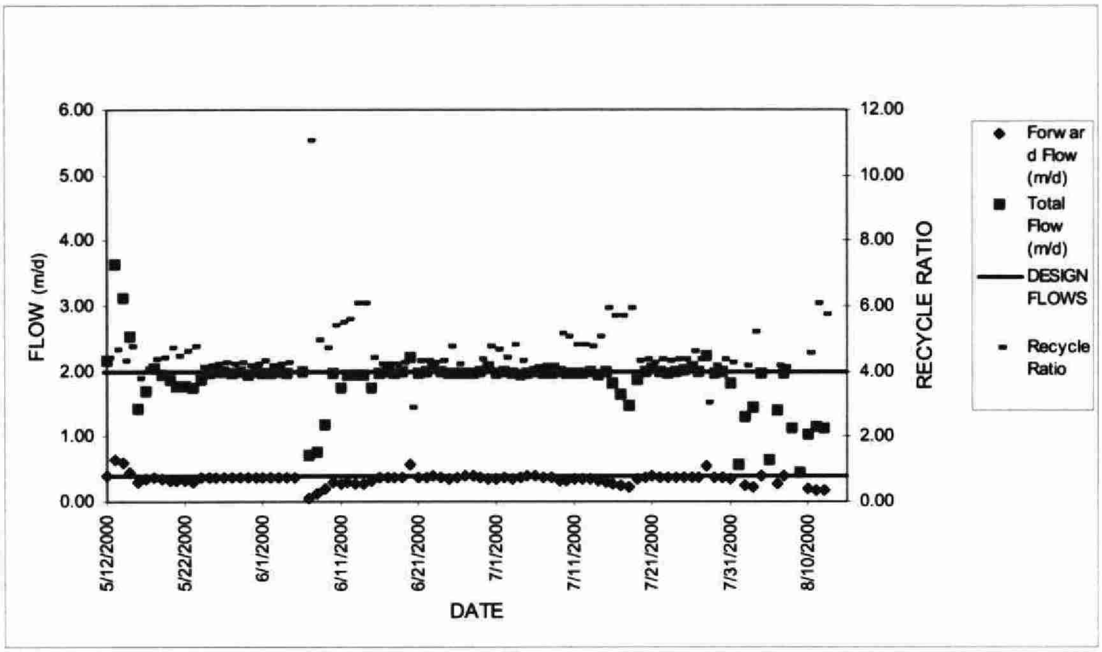

Figure AIV.13: RSF #3 Phase 2 Flows

The design values for Phase 3 were the same as those in Phase 2. The flows obtained were very close to design, as can be seen in Figure AIV.14. Near the end of Phase 3, the forward flow gradually decreased while the total flow remained the same. Consequently, the recycle ratio gradually increased.

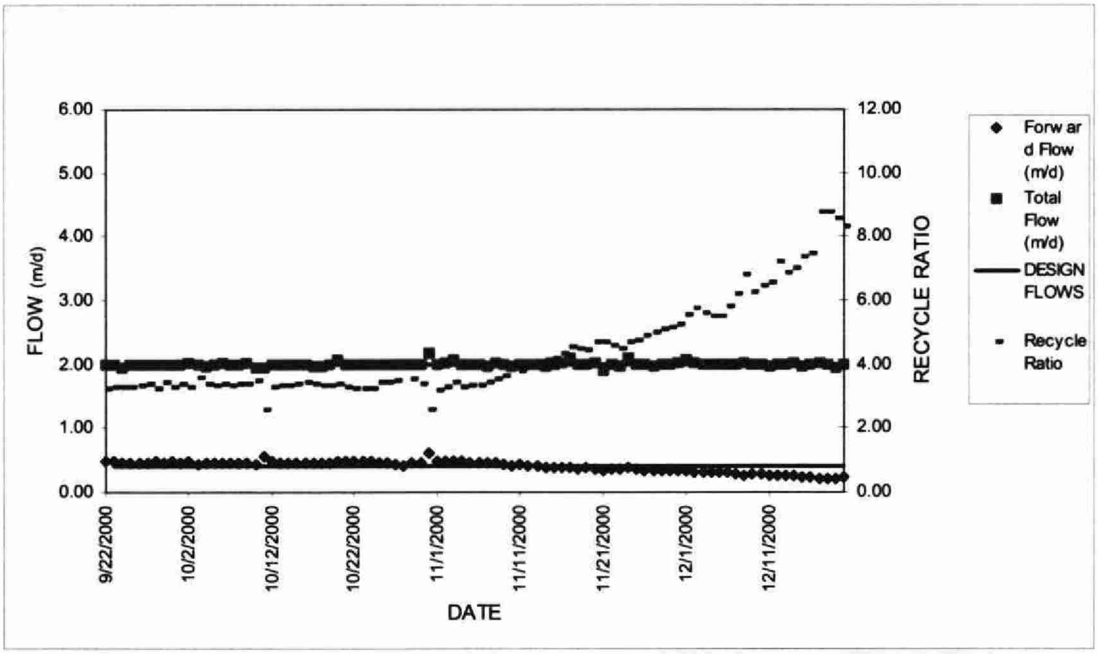

Figure AIV.14: RSF #3 Phase 3 Flows

The target settings were again left the same as in Phase 3. The wastewater and total flows were close to design, with the wastewater being slightly lower than Phase 3 and the total higher. The average recycle ratio in Phase 4 was 4.5, up from 4.1 in Phase 3. Because of

the higher that usual dosing rates for Orenco filters, the screen in the dosing tank required more frequent cleaning. It was cleaned on January 13th and again on April 19th. The wastewater flow increased both times, as can be seen in Figure AIV.15. The variability for Phase 4 was higher than in Phase 3, 0.22 vs. 0.38 for wastewater flow and 0.07 vs. 0.2 for total flow. During this phase, the wastewater flows varied from 0.17 to 1.26 m/d and the total flow from0.63 to 4.66 m/d.

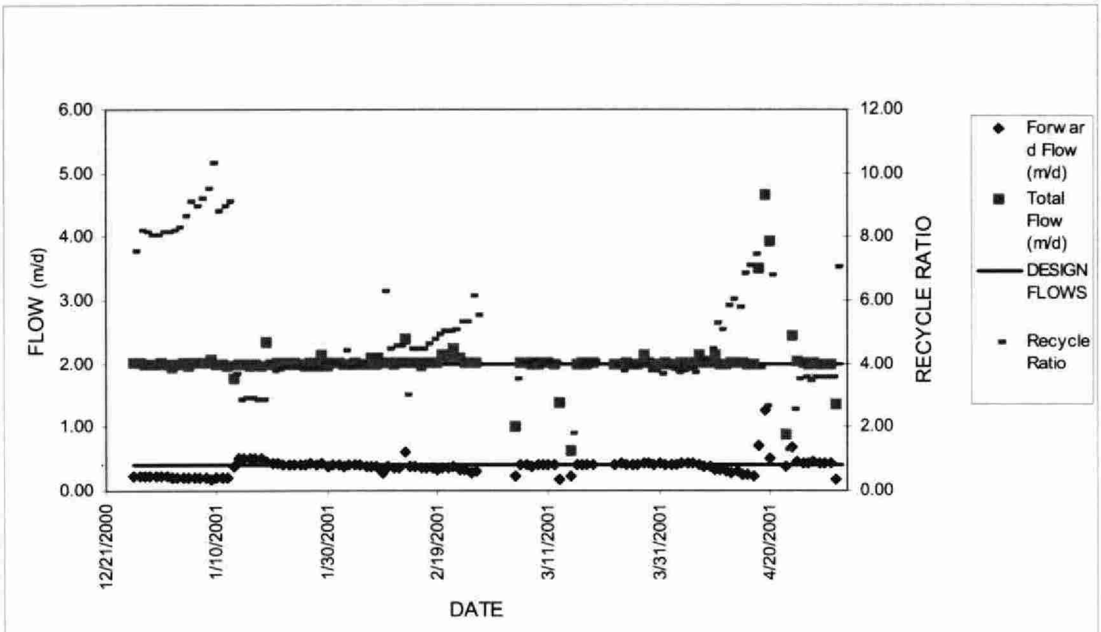

Figure AIV.15: RSF #3 Phase 4 Flows

AIV.4 RSF #4

RSF #4 was constructed with a fine media equivalent to that being used in existing Ontario sand filters. The design and actual values for flows are presented in Table AIV.4.

Table AIV.4: RSF #4 Flow Summary

Phase		Acc'n	1		2		3
Date		9/13-10/1	12/2-1/16	2/6-3/20	5/12 – 5/31	6/22 – 8/15	10/30 – 11/15
Wastewater (m/d)	Design	0.20	0.20		0.4	0.2	0.2
	Mean	0.26	0.30	0.25	0.28	0.23	0.13
	Std. Dev.	0.11	0.10	0.19	0.12	0.19	0.09
	Coef. of Var.	0.42	0.33	0.76	0.43	0.83	0.46
Total (m/d)	Design	1.00	0.60		1.2	0.6	0.6
	Mean	0.85	0.52	0.64	0.72	0.52	0.50
	Std. Dev.	0.52	0.12	0.42	0.4	0.39	0.15
	Coef. of Var.	0.61	0.23	0.66	0.56	0.75	0.30
Recycle Ratio	Design	4:1	2:1		2:1	2:1	2:1
	Mean	2.3:1	0.7:1	1.6:1	1.6:1	1.3:1	2.9:1
Dosing Frequency		1/hr	1/hr		1/hr		1/hr

RSF #4 had a very short Acclimation Phase, from September 13th to October 1st, 1999. It was removed from service on Oct 1, 1999 because of surface ponding. Prior to this date, RSF#4 had a variable applied wastewater flow (Coef. of Var. 0.42) ranging from 0.12 to 0.59 m/day (Fig. 8). The mean daily flow (0.26 m/day) was 30% higher than the design flow. Even with a consistent recycle flow from the splitter the actual total flow on occasion ranged from 0.17 to 2.58 m/d but averaged 0.85 m/day. The actual recycle ratio was only 2.3:1 or 57.5 % of design. Figure AIV.16 shows the flows and recycle ratio for RSF #4 for the Acclimation Phase.

RSF #4, with the fine media, exhibited surface ponding during and following the hourly applications of wastewater and recycled RSF effluent. When the filter surface had not drained prior to the next application it was deemed that the hydraulic capacity of this fine media was insufficient for hourly application of the mean applied hydraulic loading (1.0 m/day). Ponding would undoubtedly cause freezing problems during cold weather operation. The original media, with an Effective Size ($ES=D_{10}$) of 0.10 and a Uniformity Coefficient ($UC=D_{60}/D_{10}$) of 2.4, while typical of ISFs in Ontario, was somewhat finer than the media used for the ISFs at the Clifford STP (ES = 0.15 and UC = 3.9). A decision was made to replace the media with a second somewhat coarser fine media (ES = 0.14 and UC = 6.4). Flow was terminated to RSF #4 on October 2nd. At the same time as the media replacement, the geotextile cloth covering the underdrains was replaced.

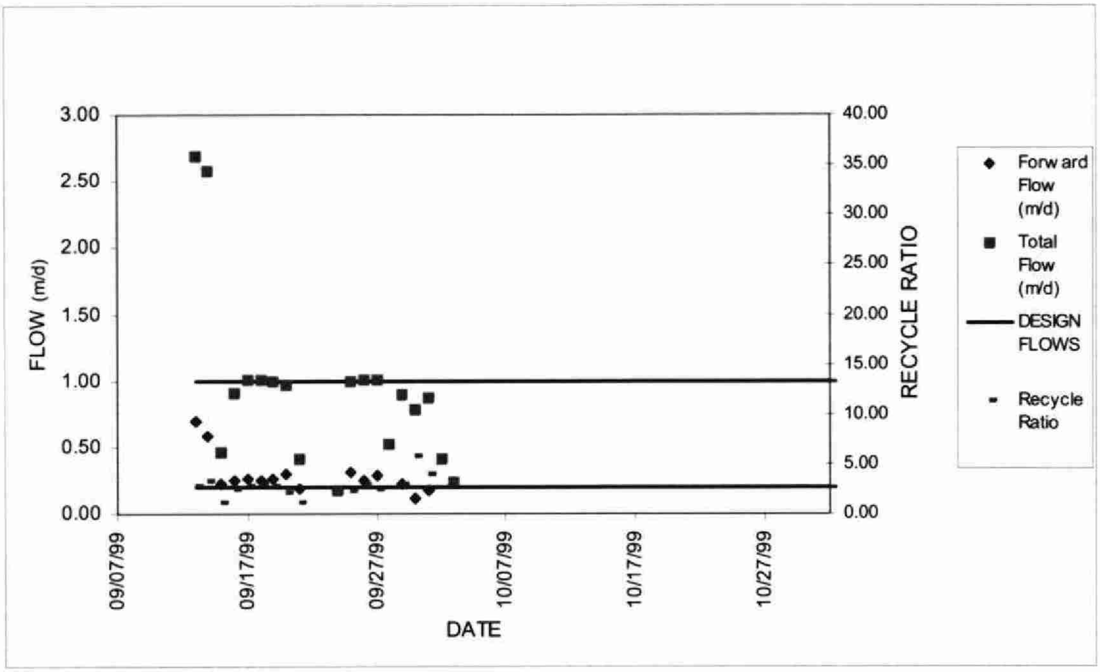

Figure AIV.16: RSF #4 Acclimation Flows

Phase 1 for RSF #4 commenced on December 2nd, 1999. RSF #4 with the new media demonstrating improved hydraulic conductivity and reduced ponding. However, with the onset of cold weather, even this reduction in ponding did not prevent the surface of the filter freezing around January 7th, as shown in Fig. AIV.17. Surface ponding caused the RSF#4 to freeze, terminating the run on January 7th. On January 19th attempts were made to thaw RSF #4 and on February 2nd it was placed back in service until March 20th, when ponding again occurred and the phase was terminated. Wastewater flows to RSF#4 were extremely erratic for both periods ranging from 0.09 to 0.83 m/day. The average wastewater flow over the period prior to freezing was 0.30 m/day or 150 % of the design value. For the part of the phase after thawing, the average wastewater flow was 0.25 m/d, or 125 % of design. Over the entire phase, the total applied flows ranged from 0.05 to 2.61 m/day, with a mean value of 0.52 m/day or 87 % of the design value prior to freezing and 0.64 m/d or 107% of design. The average recycle ratio prior to freezing was only 0.7:1 or 35 % of design. After freezing the recycle ratio increased to 1.6:1, or 80 % of design. Faulty pressure transducers were identified by SFI as the cause of the variability in the flows.

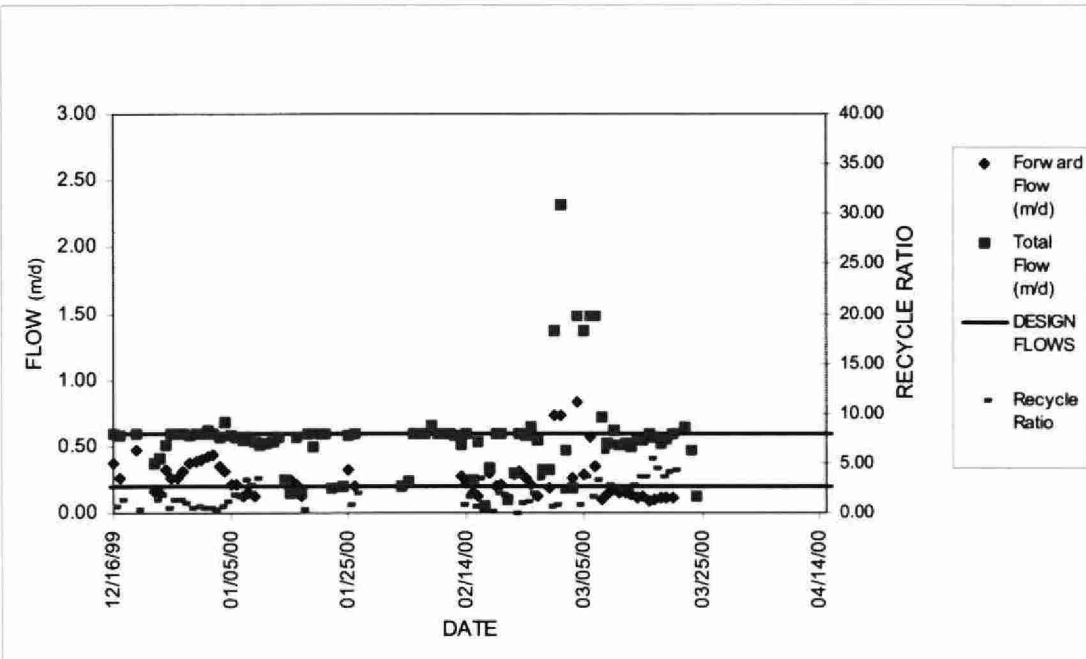

Figure AIV.17 RSF #4 Phase 1 Flows

After March 20th, RSF !4 was allowed to drain and dry out before the commencement of Phase 2. During Phase 2, RSF #4 still experienced hydraulic problems. Initially, the design flow was 0.4 m/d and the actual flow averaged out to 0.28 m/d over the period of May 12th to June 5th, 2000. By May 28th, ponding had commenced and the flow was terminated on June 5th. Since there appeared to be no biological reason for this ponding (i.e. biological mat on surface) the filter was allowed to dry out again and the settings for design flow were lowered to 0.2 m/d with a corresponding reduction in total flow to 0.6 m/d from 1.2 m/d. Flow was restarted June 22nd but equipment problems were encountered and the flow was again turned off on July 4th. Another attempt was made to operate RSF #4 during August, but problems with the main wet well precluded longer operation. During the second part of Phase 2, the wastewater flow averaged 0.23 m/d and the total flow 0.52 m/d. The high Coef. Of Var. for both parts of this phase reflect the erratic behaviour of the flows to this filter.

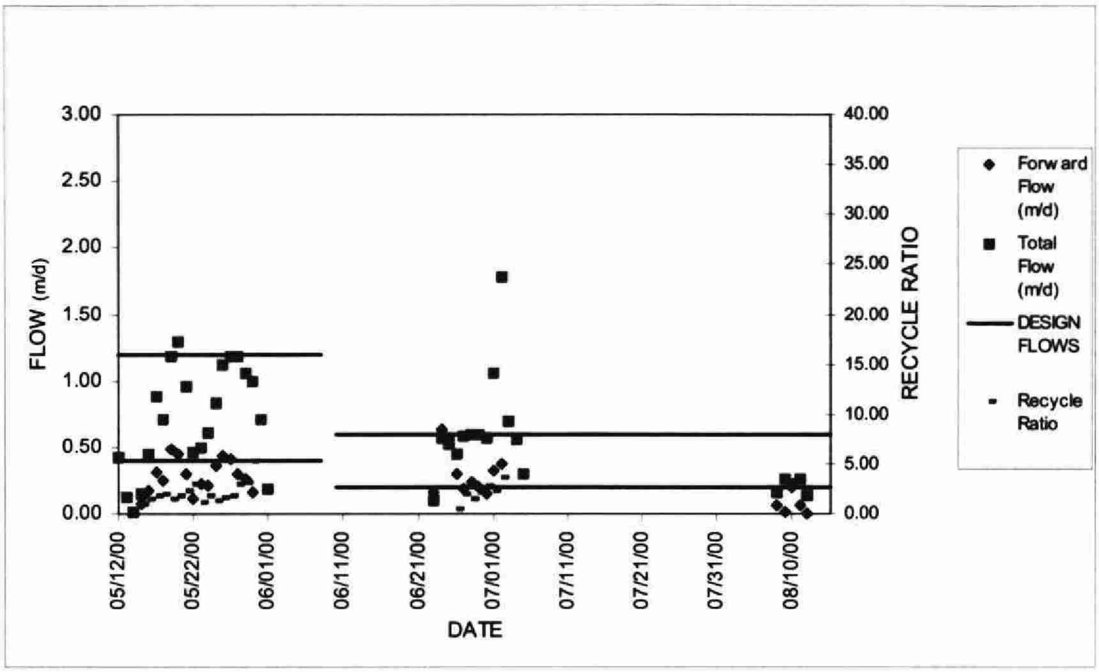

Figure AIV.18: RSF #4 Phase 2 Flows

RSF #4 was again rested and Phase 3 commenced on October 30th Initially, the flows were close to design but equipment problems again caused drifting. These problems could not be resolved and with the onset of the cold weather, the filter was shut off on November 15th. For the short period of operation, the average wastewater flow was 0.13 m/d or 65% of design and the total flow was 0.50 or 83% of design. With the low wastewater flow, the recycle ratio was 2.9:1 (145% of design).

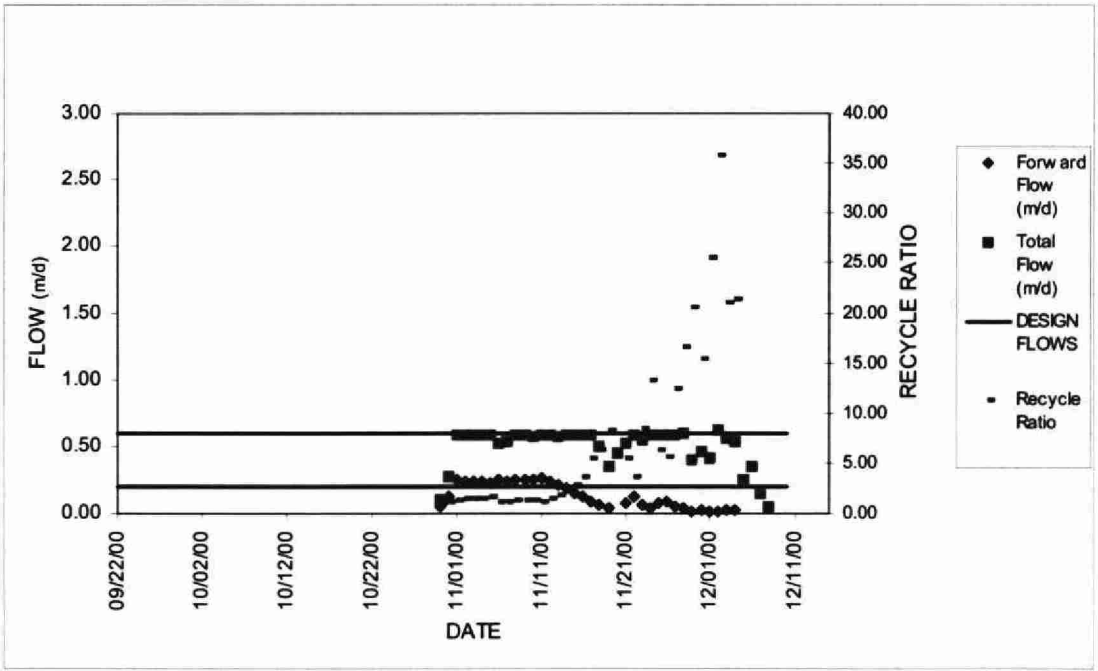

Figure AIV.19: RSF #4 Phase 3 Flows

The problems with RSF #4 appeared to be hydraulic. Analysis of the sand taken during the break in Phase 2 showed no biological reason for failure. The actual cause of the problem could not be determined, and with the history of ponding for this filter, and the onset of cold weather, it was decided that RSF #4 should not be operated during Phase 4.

APPENDIX V: LOADING SUMMARIES

The following sections contain graphs for each RSF and each phase of pilot plant operation, showing the mass loading rate and resulting effluent concentration. The graphs included are for $CBOD_5$, TSS, ammonia + ammonium, TN and finally, total nitrogen applied and the effluent nitrate + nitrate.

A.V 1.0 RSF#1 Loading

A.V 1.1 RSF #1 Acclimation

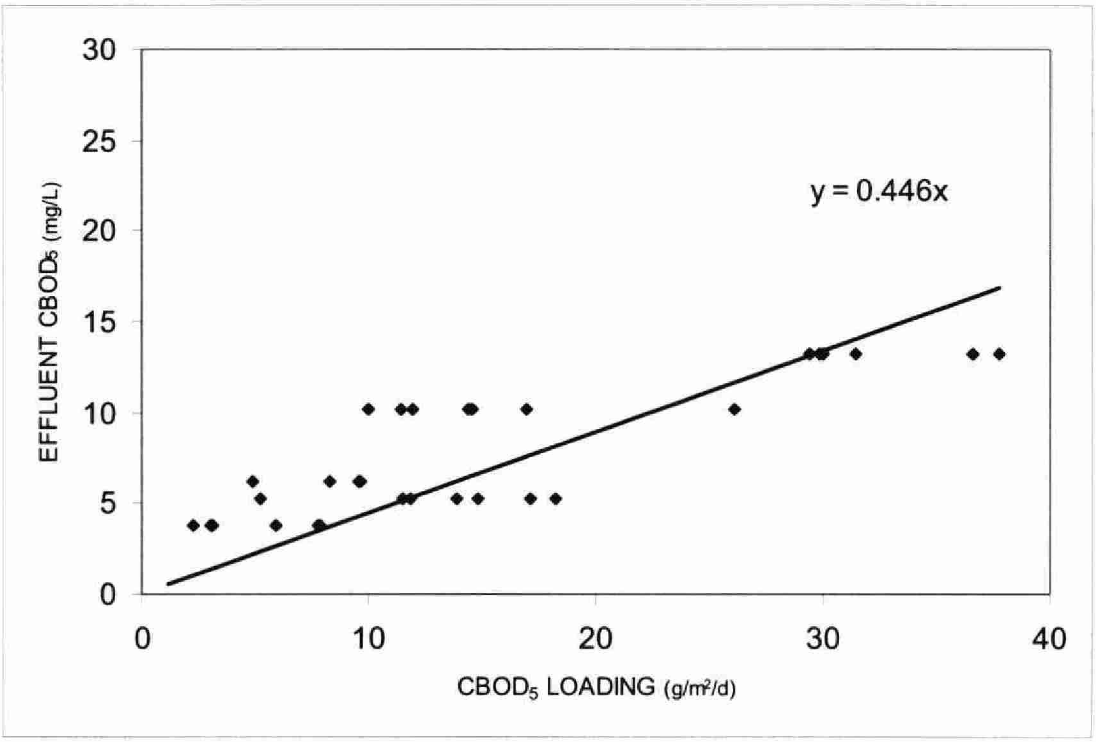

Figure A.V 1.1: RSF #1 Acclimation Phase $CBOD_5$ Loading vs. Effluent $CBOD_5$

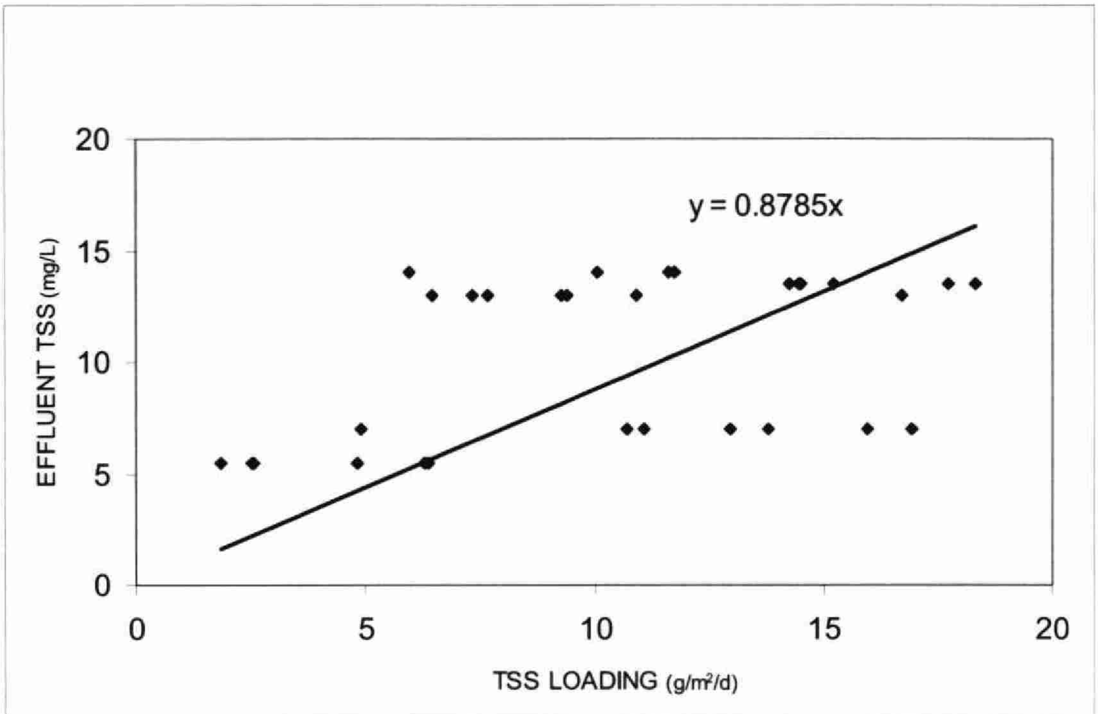

Figure A.V 1.2: RSF #1 Acclimation Phase TSS Loading vs. Effluent TSS

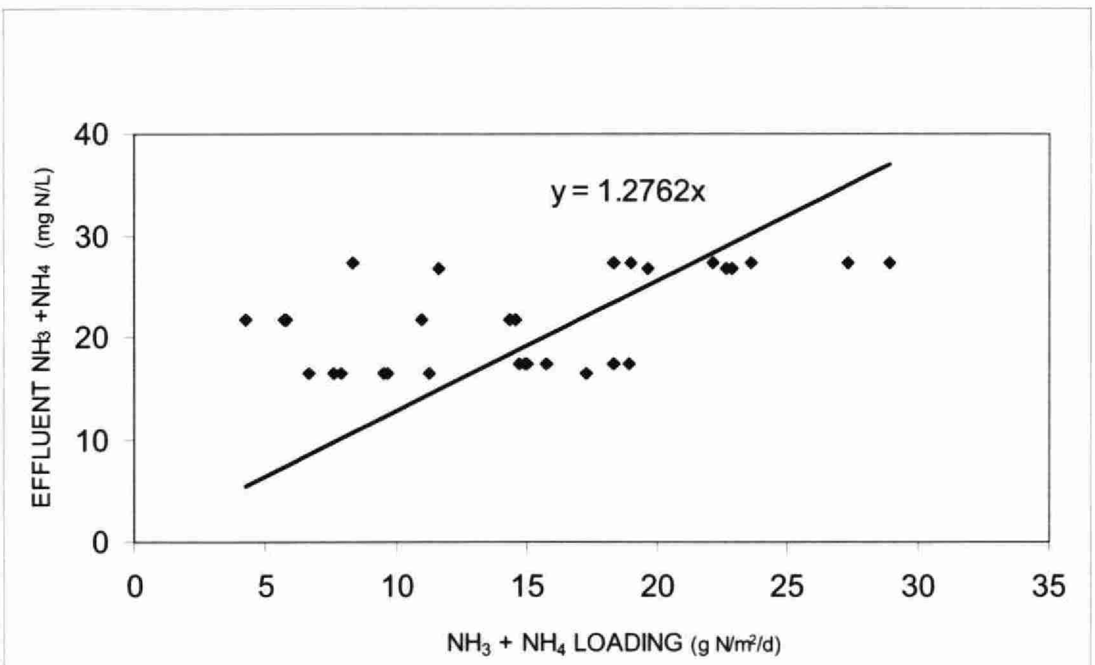

Figure A.V 1.3: RSF #1 Acclimation Phase NH_3 +NH_4 Loading vs. Effluent NH_3 +NH_4

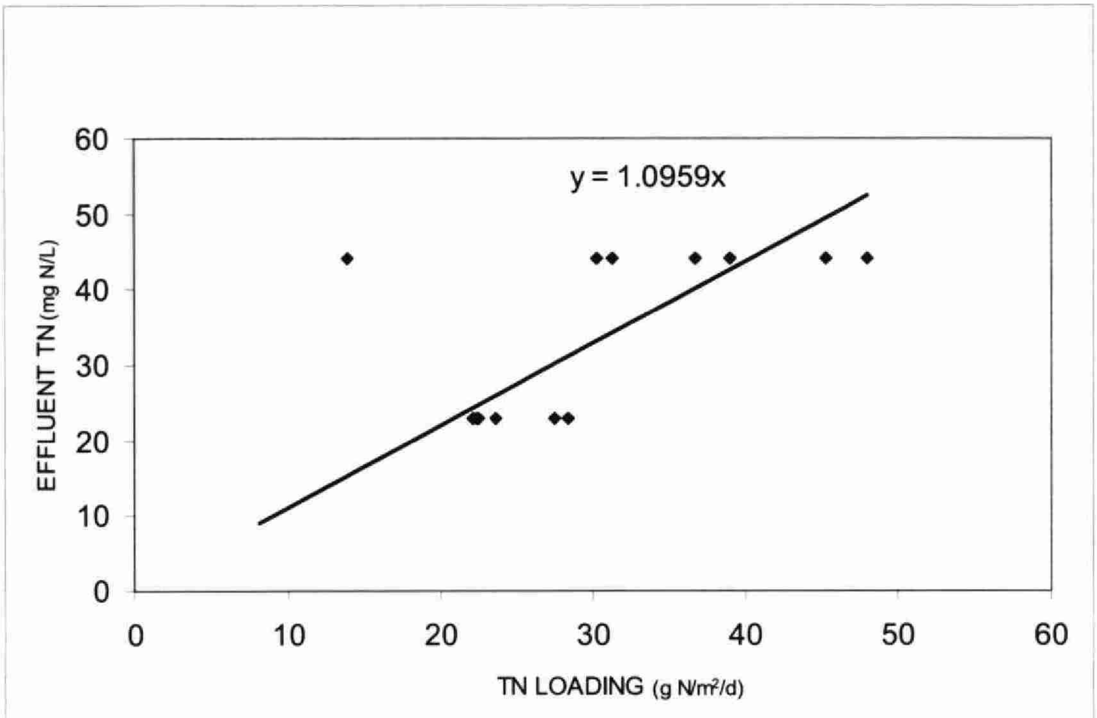

Figure A.V 1.4: RSF #1 Acclimation Phase TN Loading vs. Effluent TN

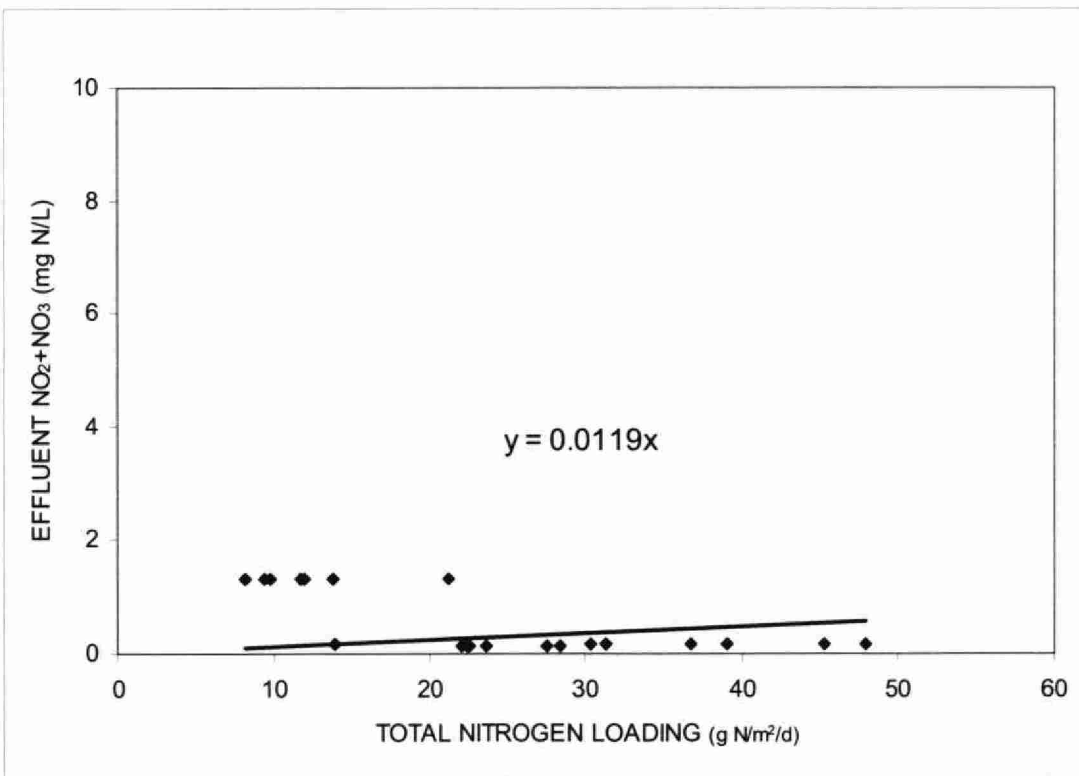

Figure A.V 1.5: RSF #1 Acclimation Phase TN Loading vs. Effluent $NO_2 + NO_3$

AV 1.2 RSF #1 Phase 1

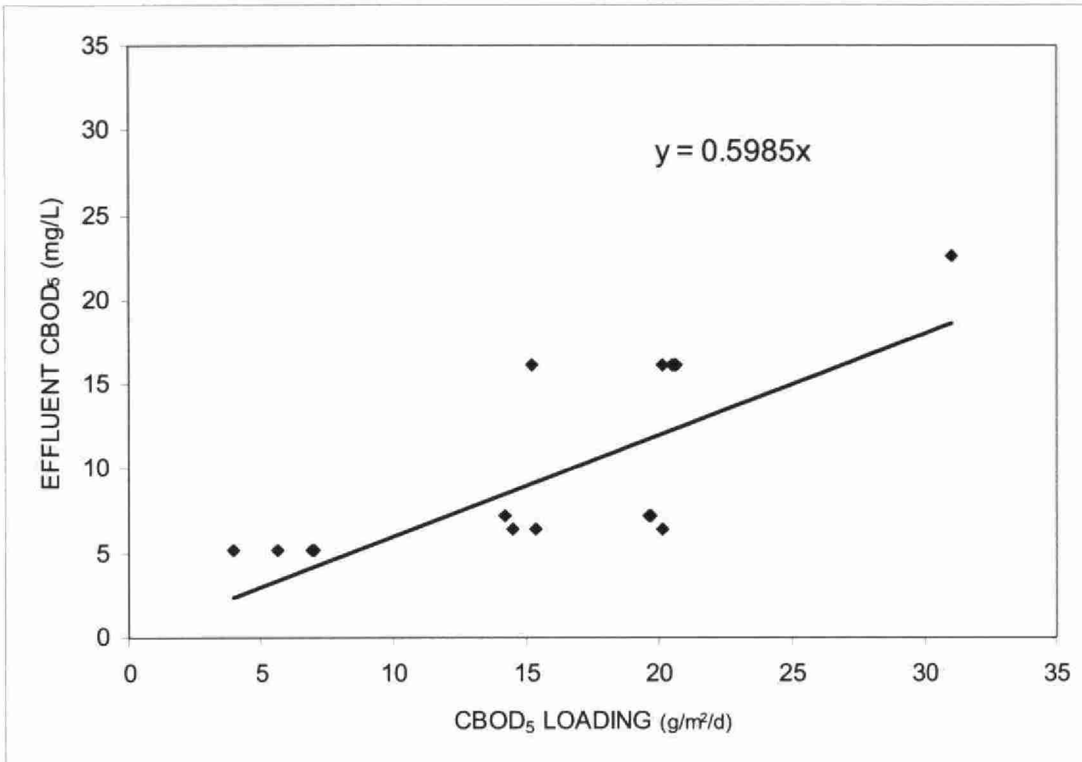

Figure A.V 1.6: RSF #1 Phase 1 $CBOD_5$ Loading vs. Effluent $CBOD_5$

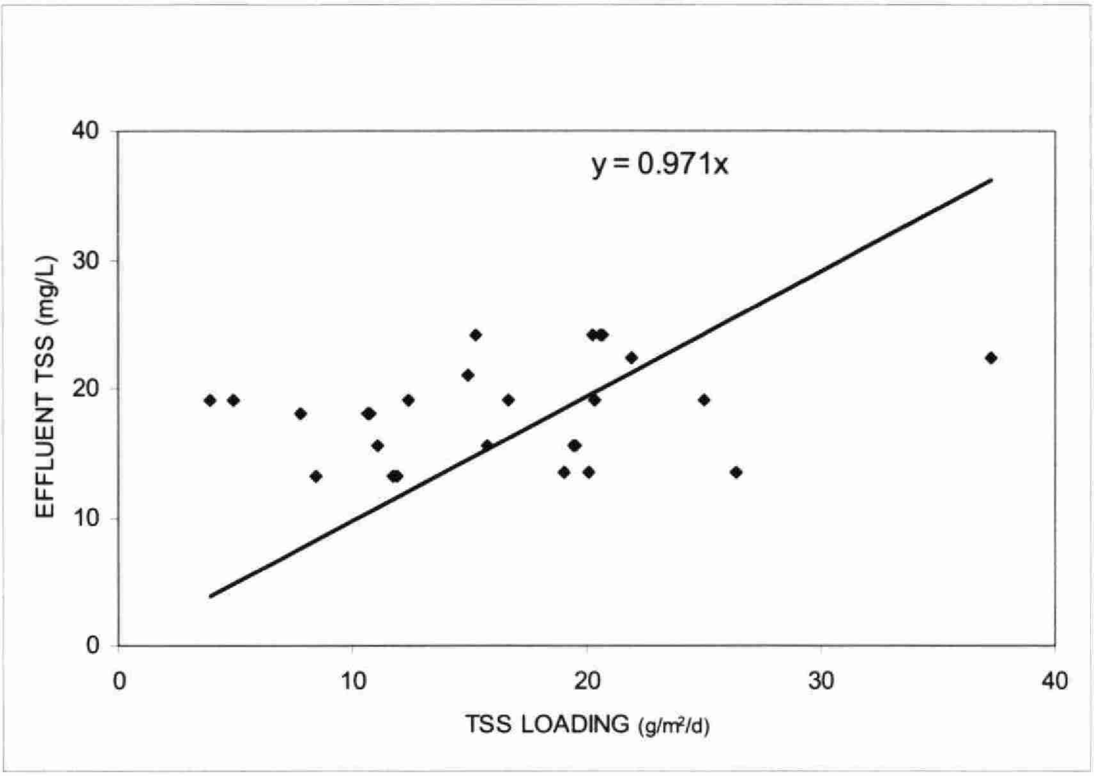

Figure A.V 1.7: RSF #1 Phase 1 TSS Loading vs. Effluent TSS

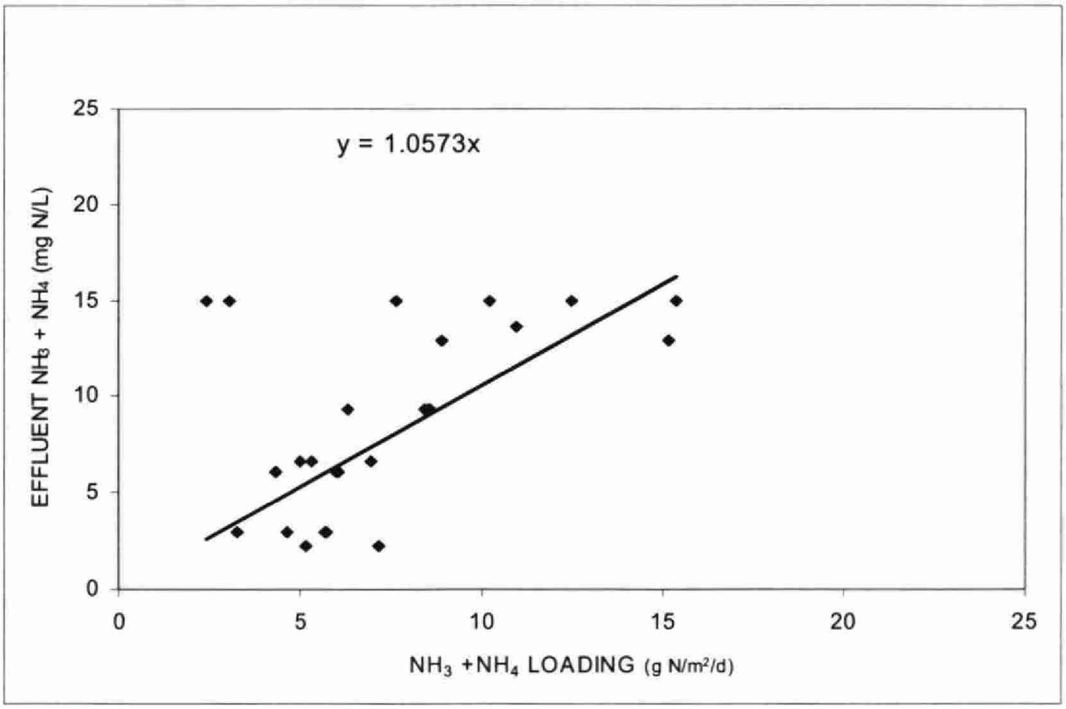

Figure A.V 1.8:RSF #1 Phase 1 NH_3 +NH_4 Loading vs. Effluent NH_3 +NH_4

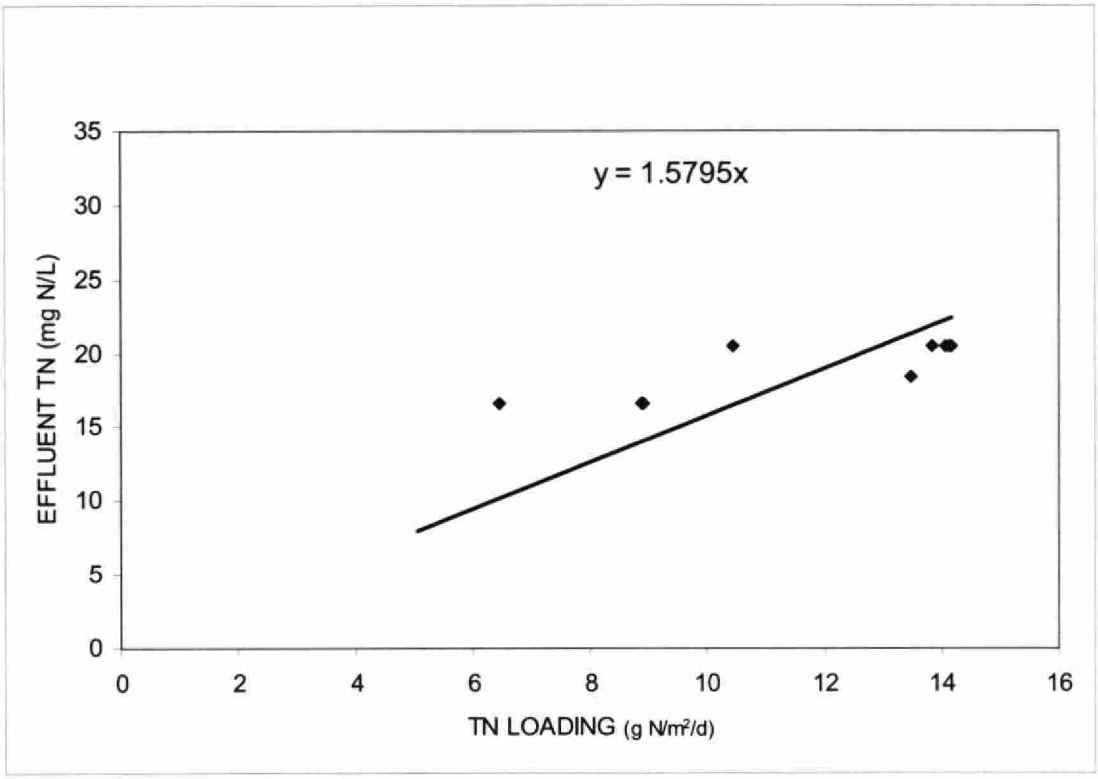

Figure A.V 1.9: RSF #1 Phase 1 TN Loading vs. Effluent TN

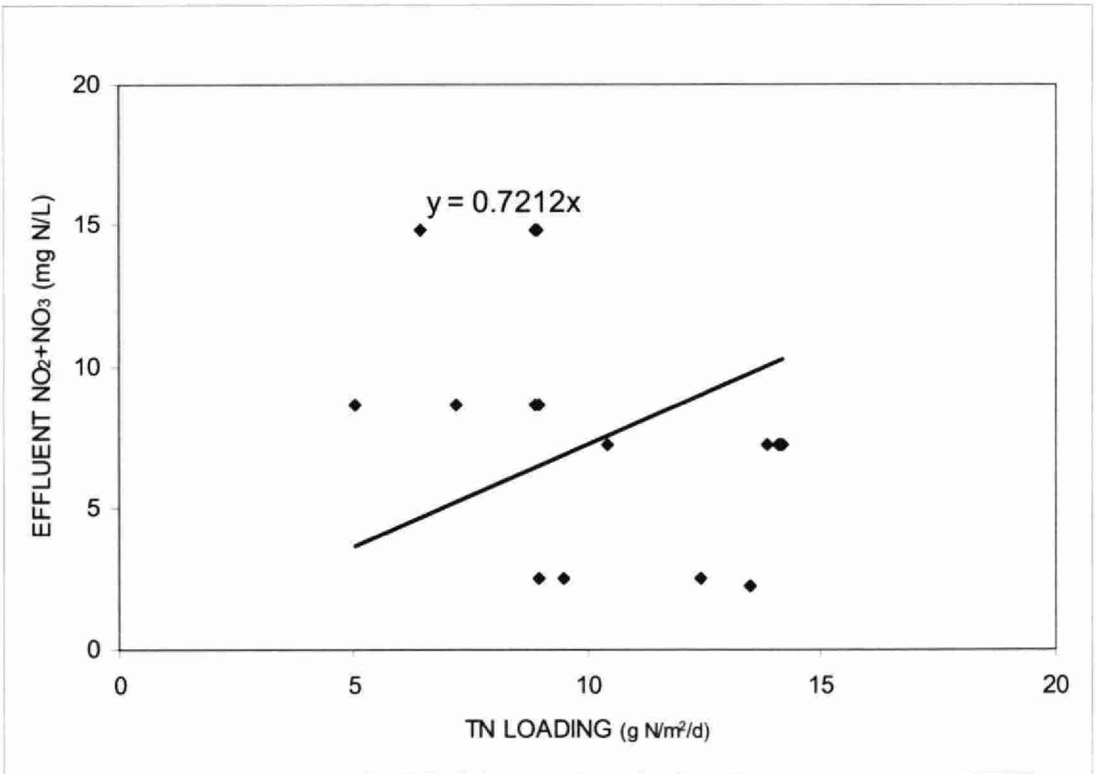

Figure A.V 1.10: RSF #1 Phase 1 TN Loading vs. Effluent $NO_2 + NO_3$

A.V 1.3 RSF #1 Phase 2

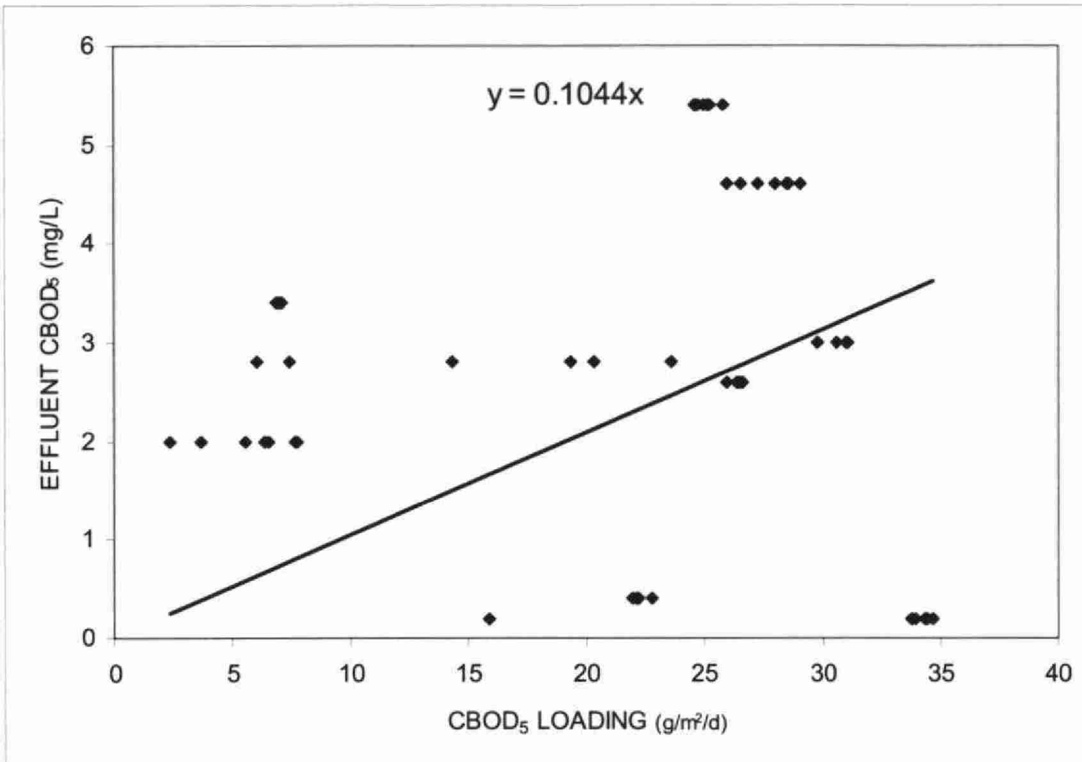

Figure A.V 1.11: RSF #1 Phase 2 $CBOD_5$ Loading vs. Effluent $CBOD_5$

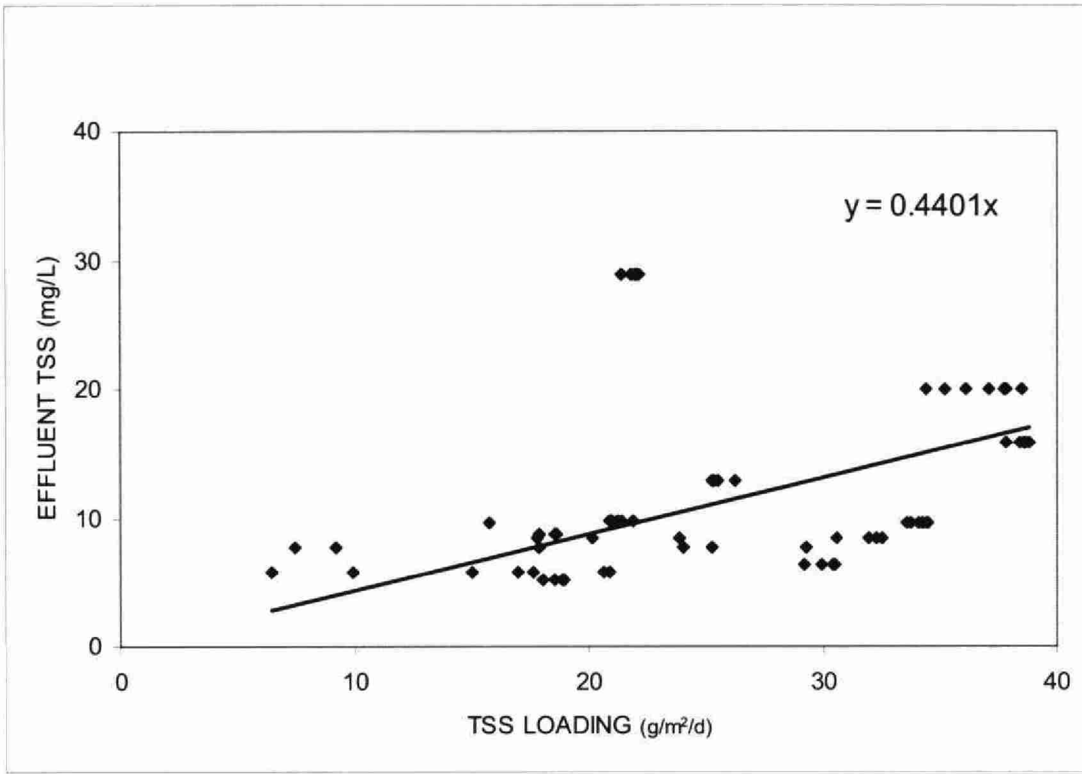

Figure A.V 1.12: RSF #1 Phase 2 TSS Loading vs. Effluent TSS

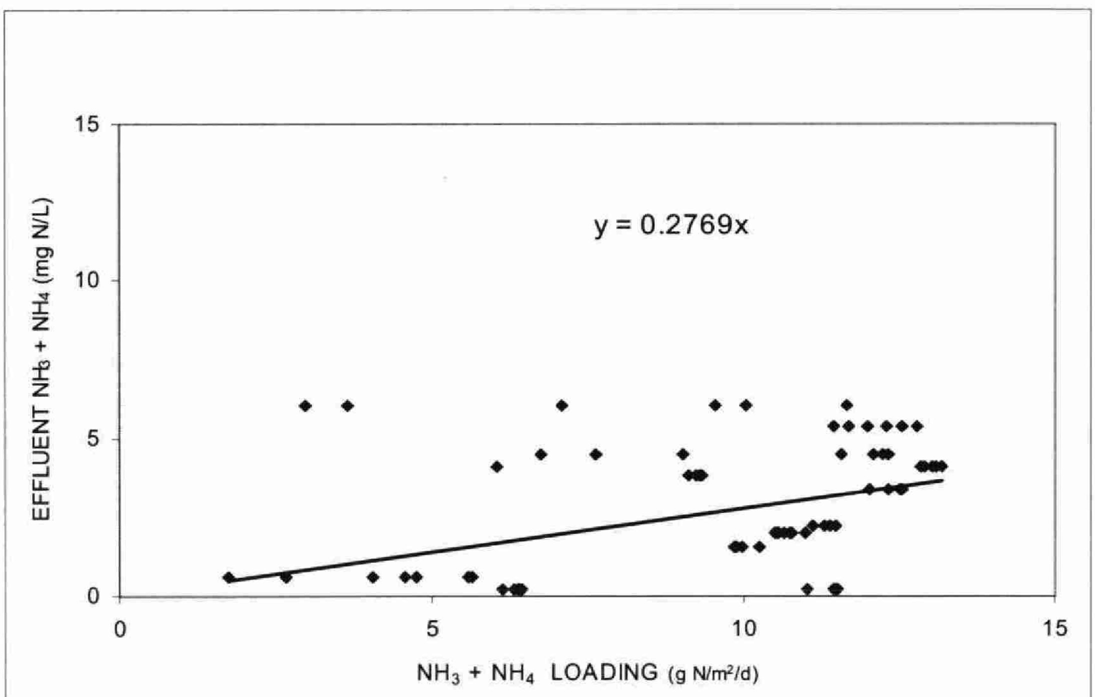

Figure A.V 1.13:RSF #1 Phase 2 NH_3 +NH_4 Loading vs. Effluent NH_3 +NH_4

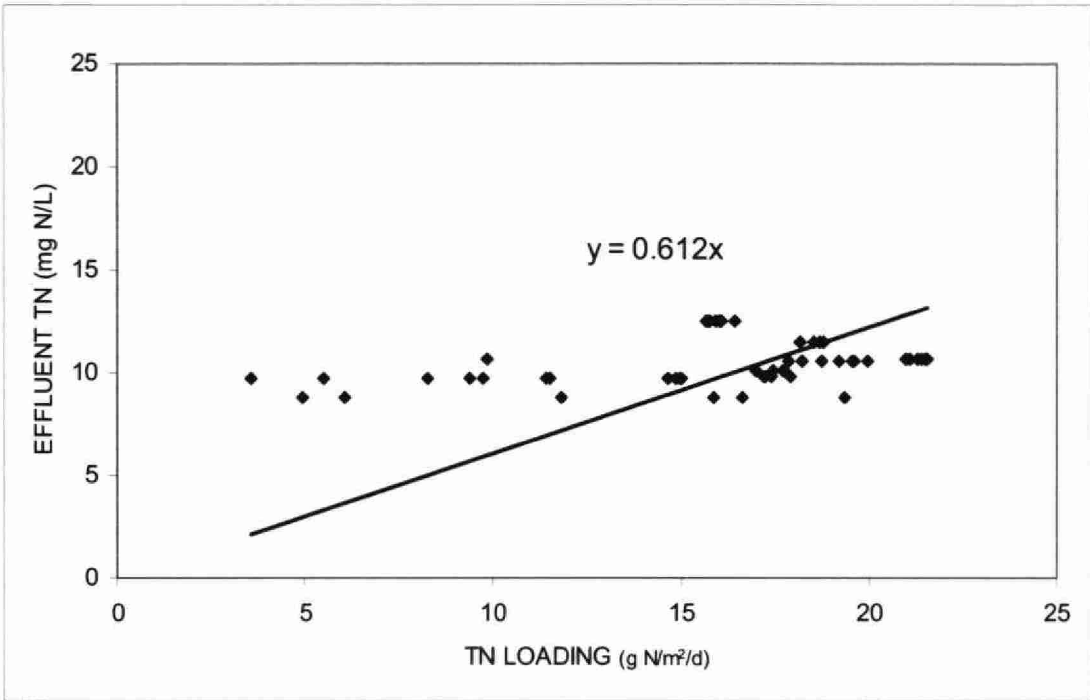

Figure A.V 1.14: RSF #1 Phase 2 TN Loading vs. Effluent TN

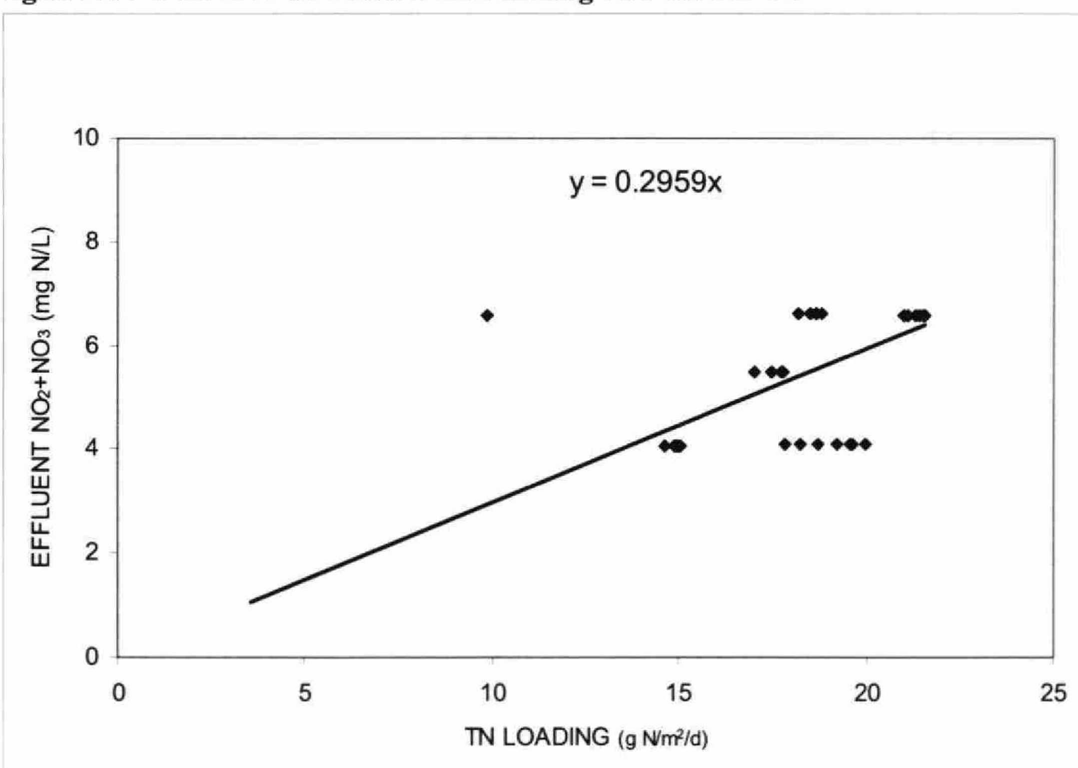

Figure A.V 1.15: RSF #1 Phase 2 TN Loading vs. Effluent $NO_2 + NO_3$

A.V 1.4 RSF #1 Phase 3

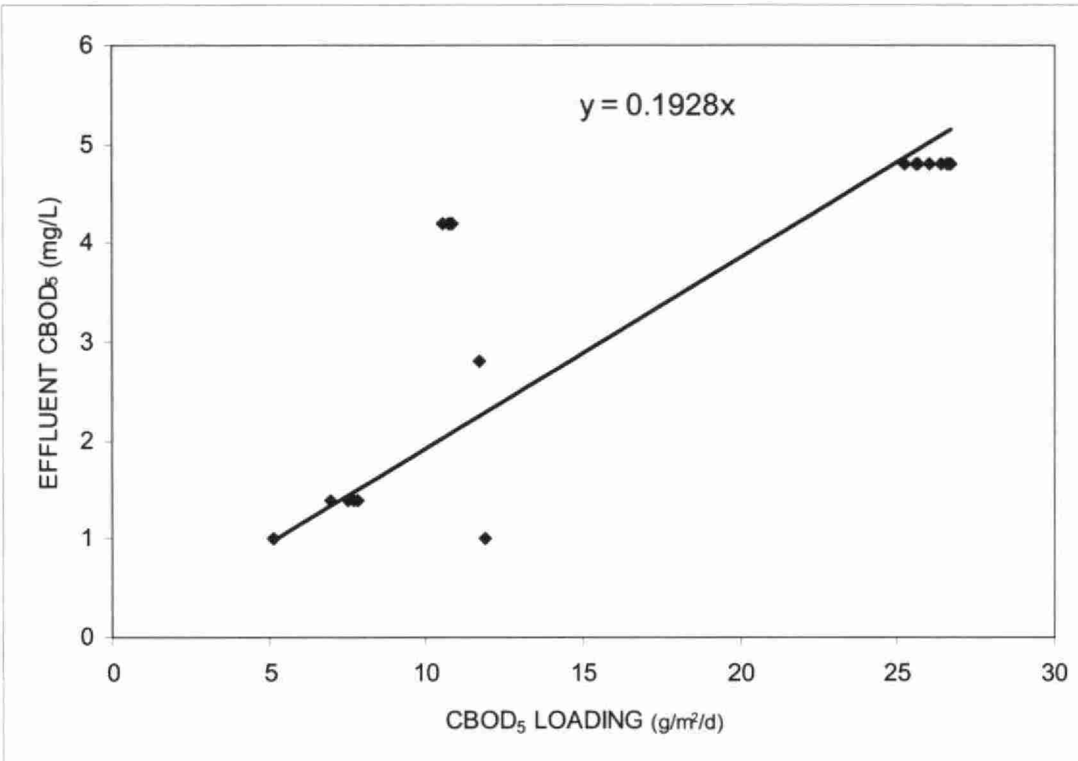

Figure A.V 1.16: RSF #1 Phase 3 $CBOD_5$ Loading vs. Effluent $CBOD_5$

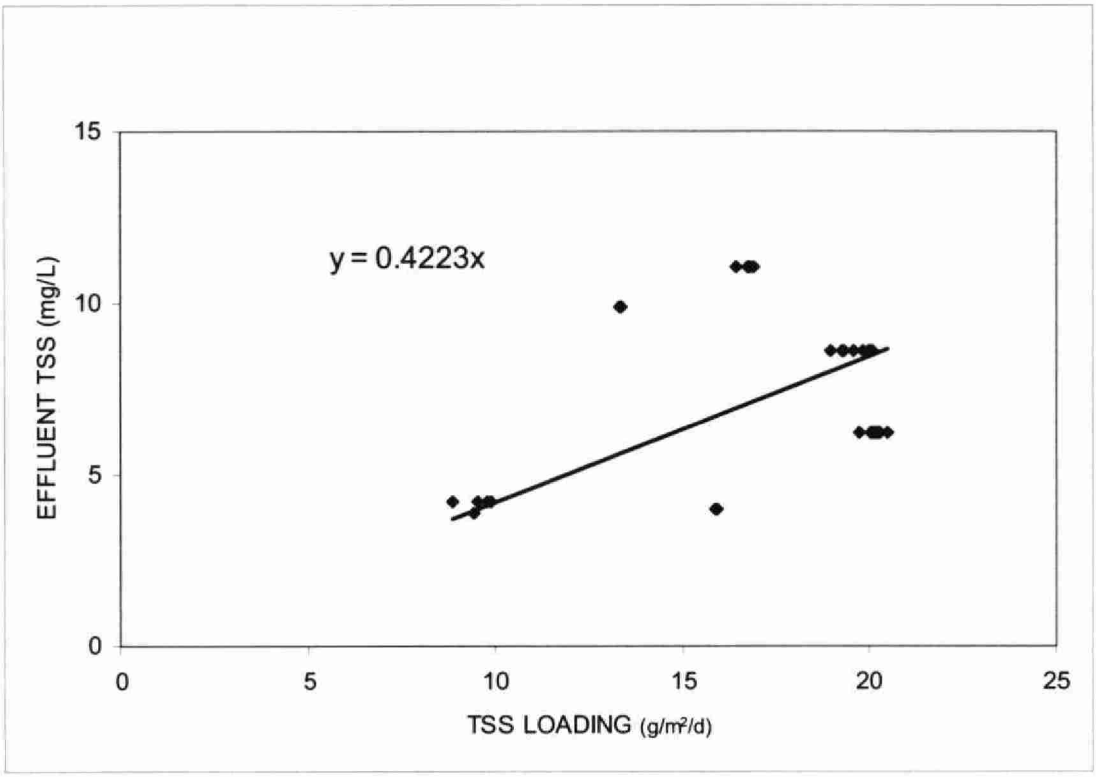

Figure A.V 1.17: RSF #1 Phase 3 TSS Loading vs. Effluent TSS

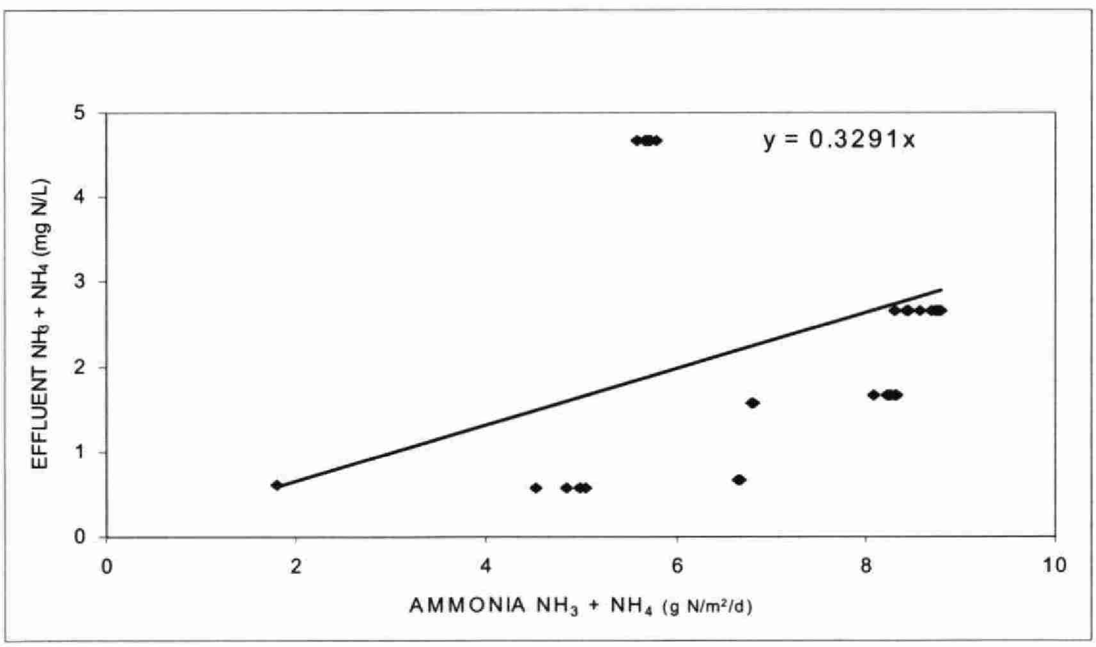

Figure A.V 1.18:RSF #1 Phase 3 NH_3 +NH_4 Loading vs. Effluent NH_3 +NH_4

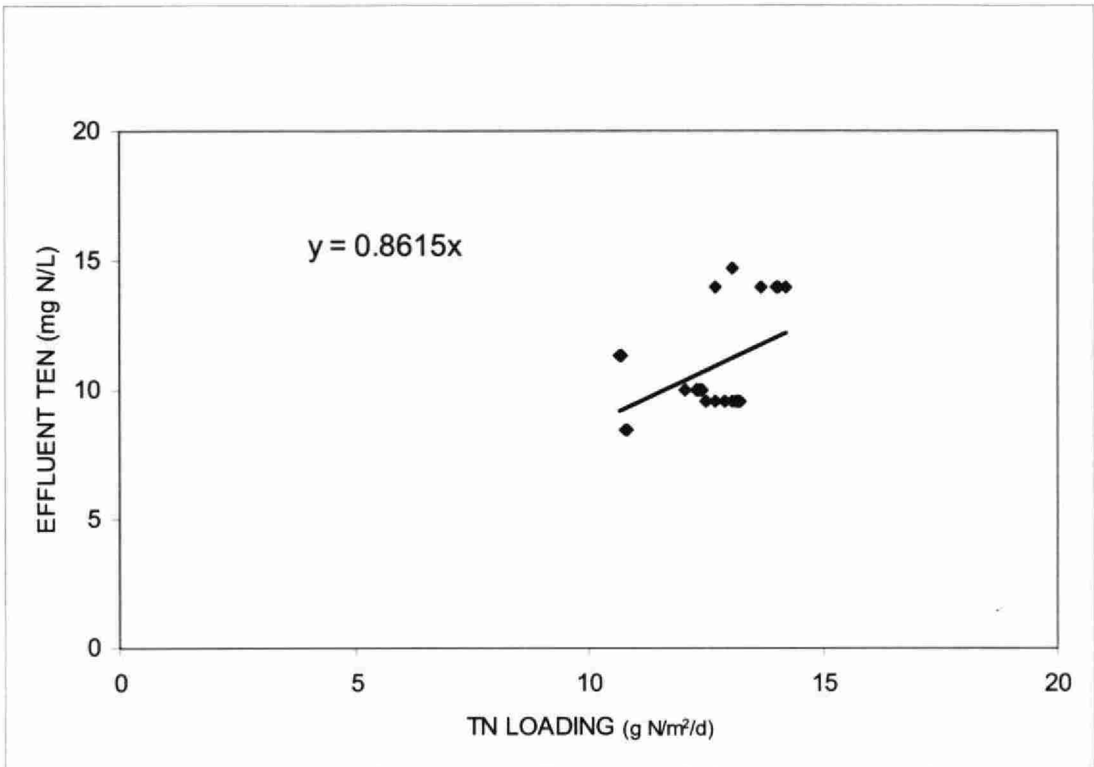

Figure A.V 1.19: RSF #1 Phase 3 TN Loading vs. Effluent TN

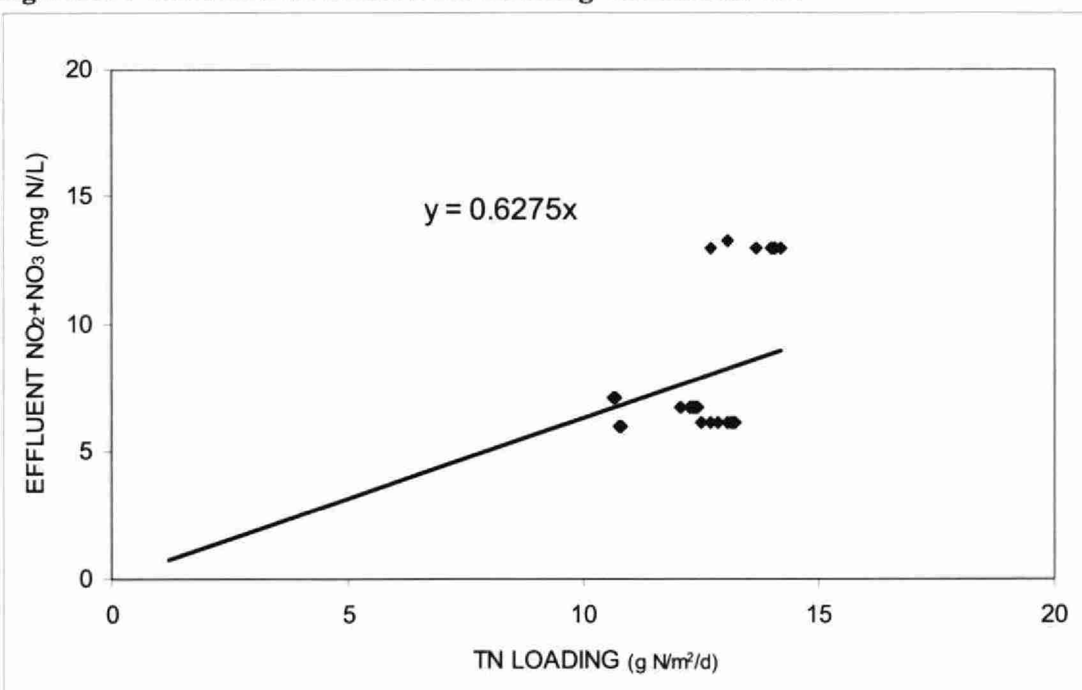

Figure A.V 1.20: RSF #1 Phase 3 TN Loading vs. Effluent NO_2 + NO_3

A.V 1.5 RSF #1 Phase 4

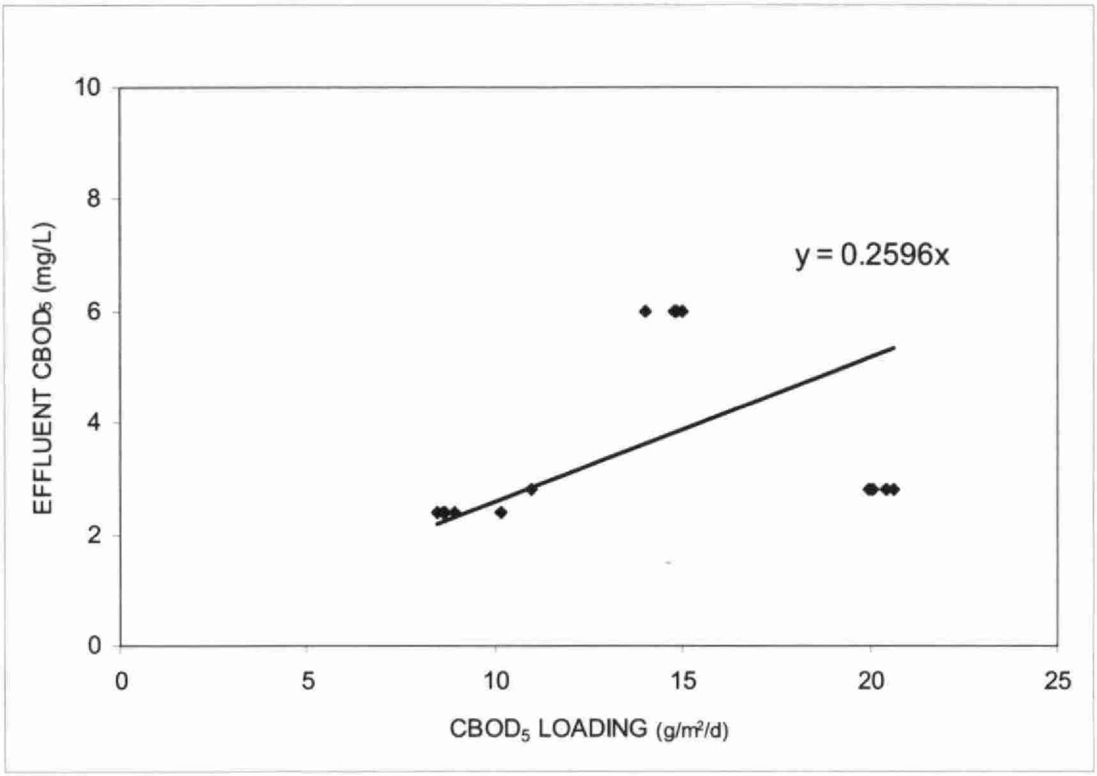

Figure A.V 1.21: RSF #1 Phase 4 CBOD$_5$ Loading vs. Effluent CBOD$_5$

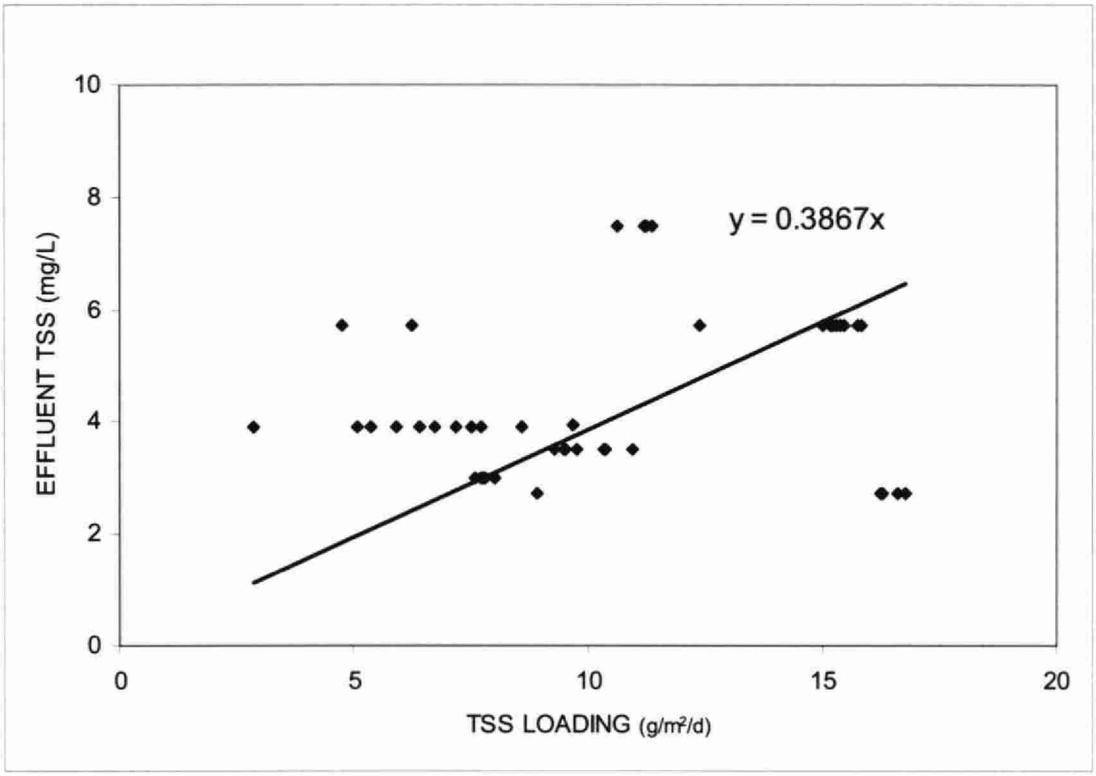

Figure .AV 1.22: RSF #1 Phase 4 TSS Loading vs. Effluent TSS

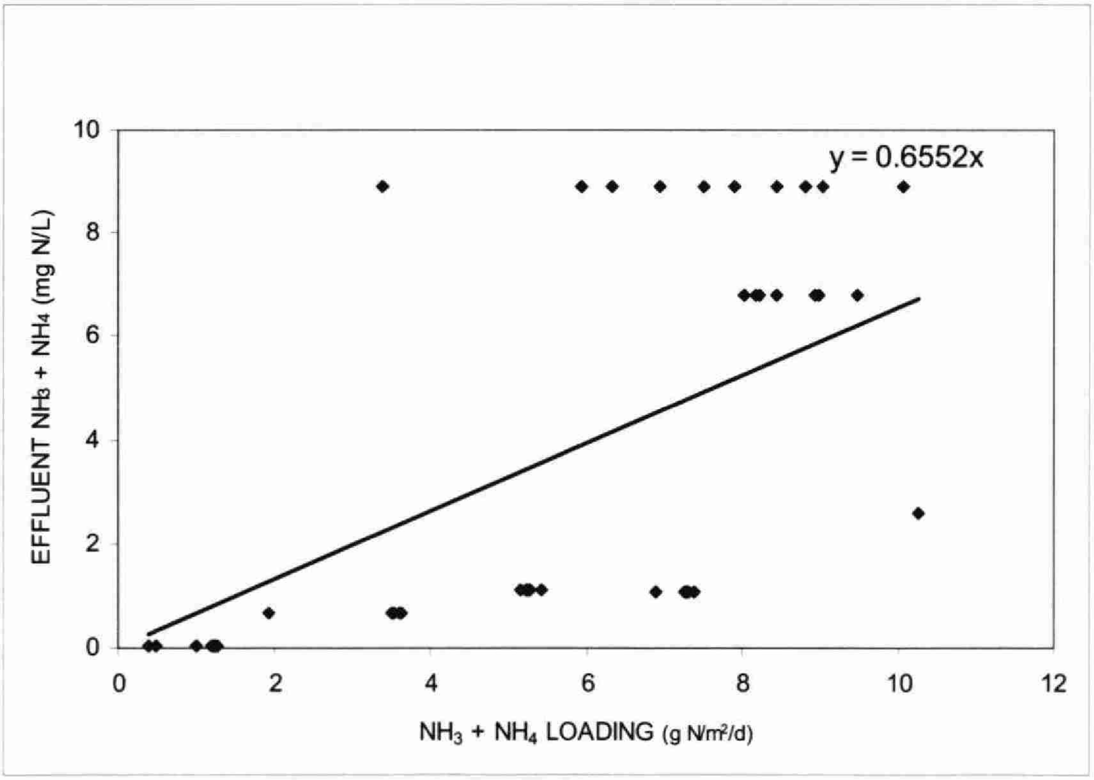

Figure A.V 1.23:RSF #1 Phase 4 NH_3 +NH_4 Loading vs. Effluent NH_3 +NH_4

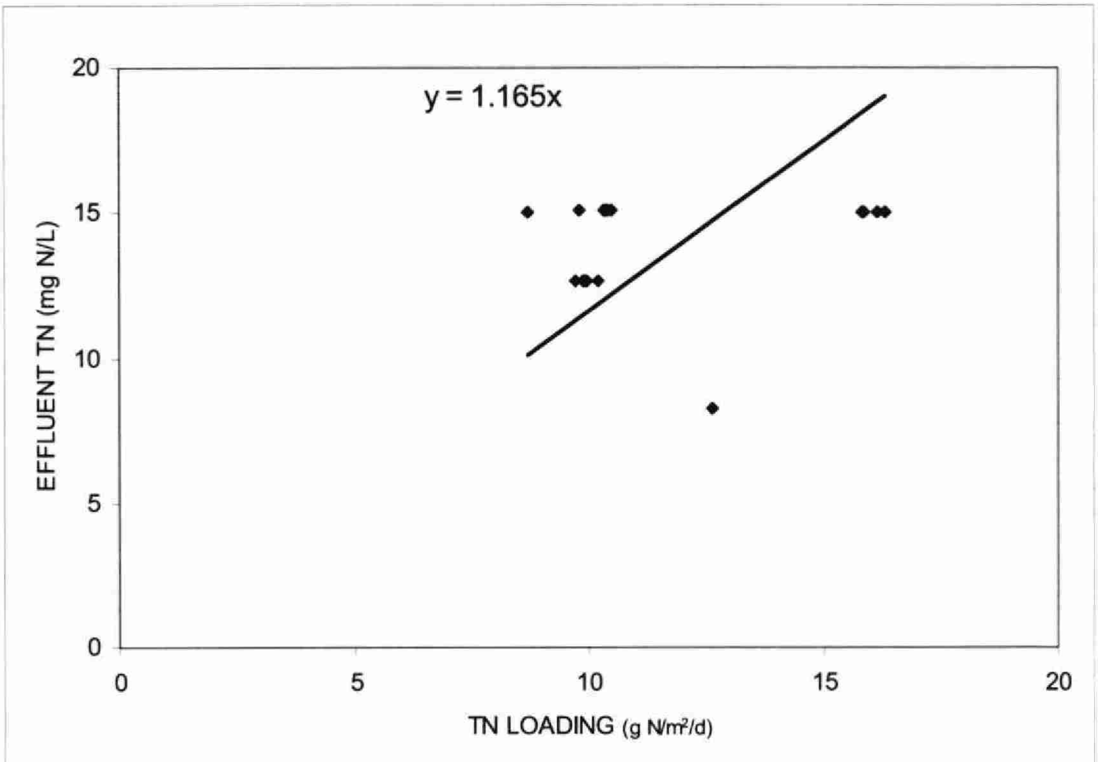

Figure A.V 1.24: RSF #1 Phase 4 TN Loading vs. Effluent TN

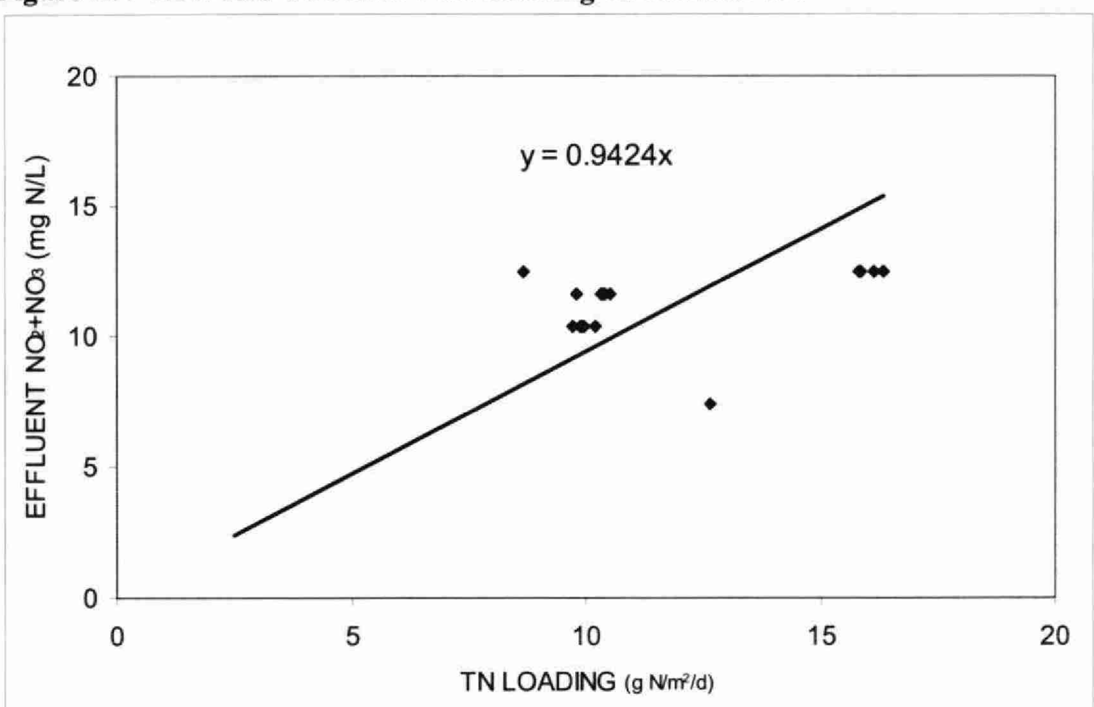

Figure A.V 1.25: RSF #1 Phase 4 TN Loading vs. Effluent NO_2 + NO_3

A.V 2.0 RSF #2 Loadings

A.V 2.1 RSF #2 Acclimation Phase

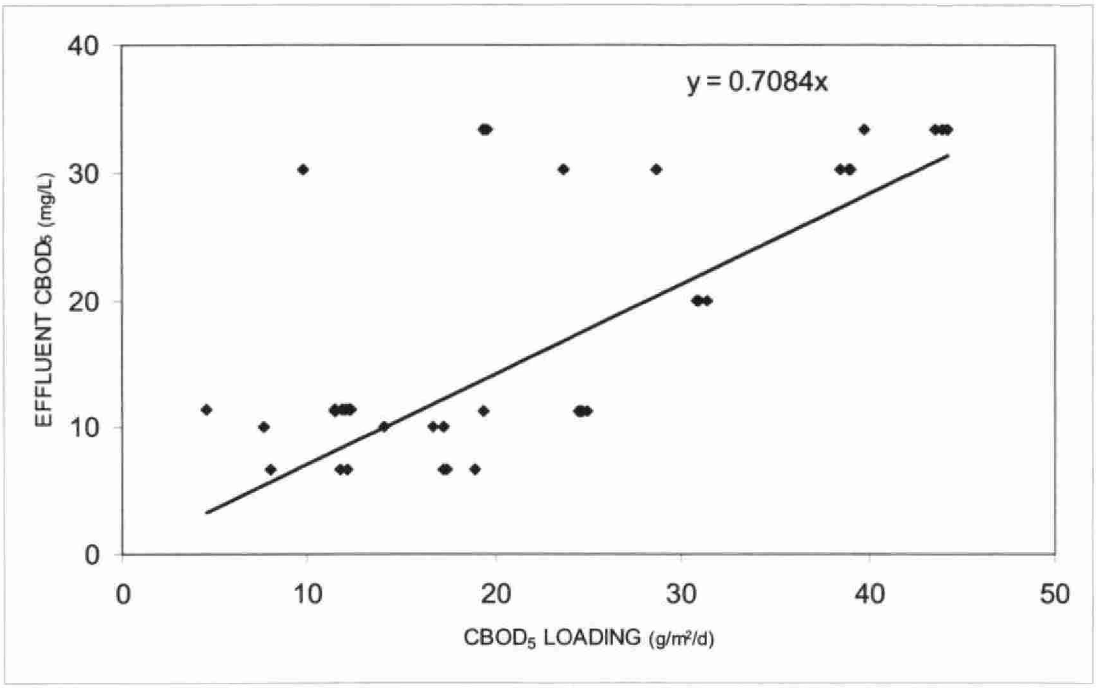

Figure A.V 2.1: RSF #2 Acclimation Phase $CBOD_5$ Loading vs. Effluent $CBOD_5$

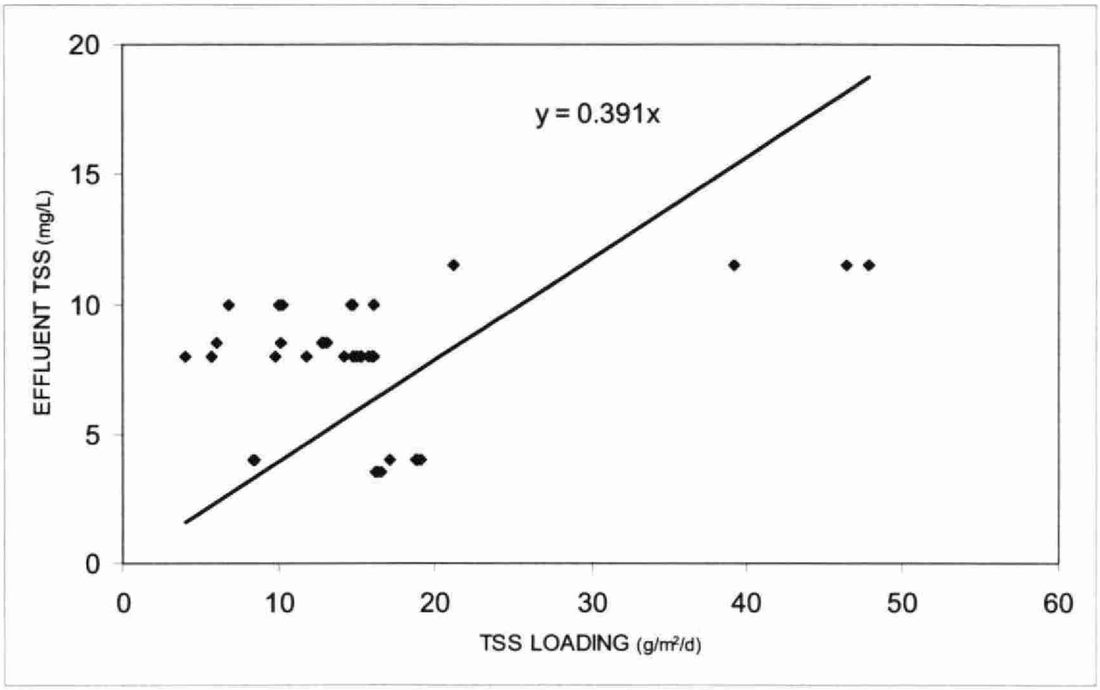

Figure A.V 2.2: RSF #2 Acclimation Phase TSS Loading vs. Effluent TSS

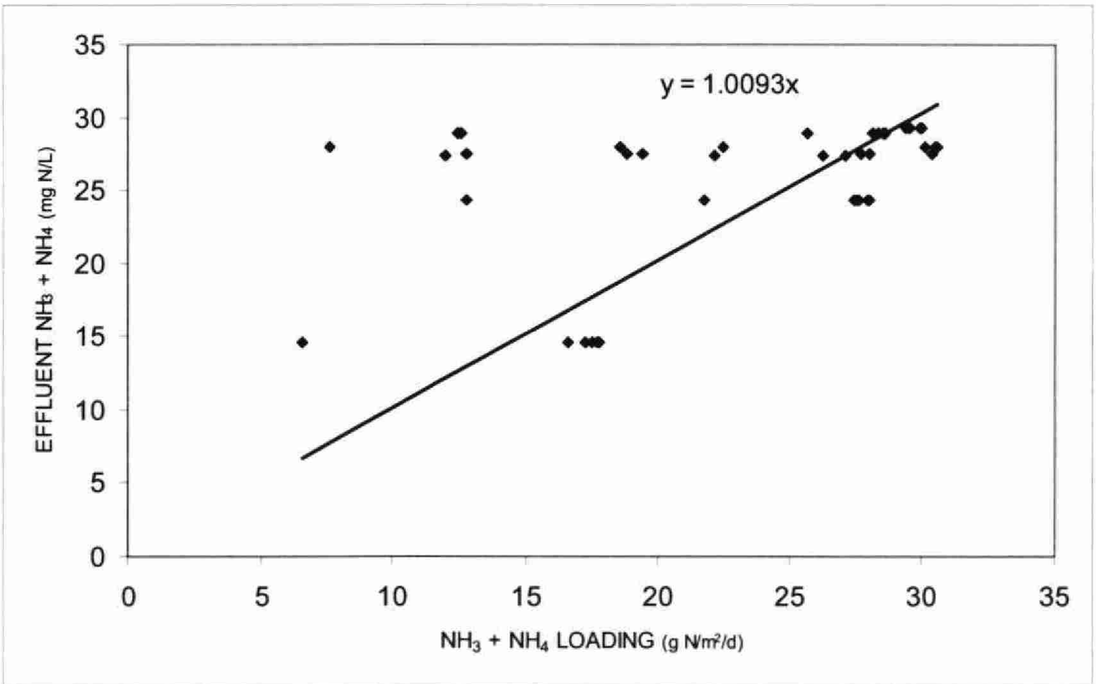

Figure A.V 2.3:RSF #2 Acclimation Phase NH_3 +NH_4 Loading vs. Effluent NH_3 +NH_4

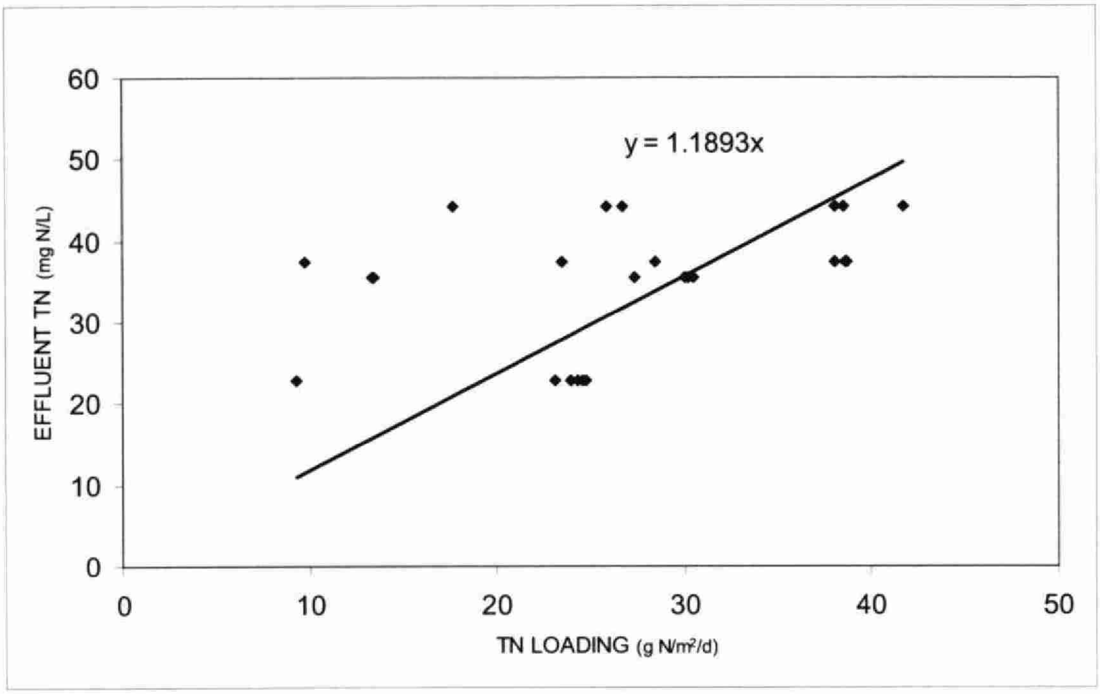

Figure A.V 2.4: RSF #2 Acclimation Phase TN Loading vs. Effluent TN

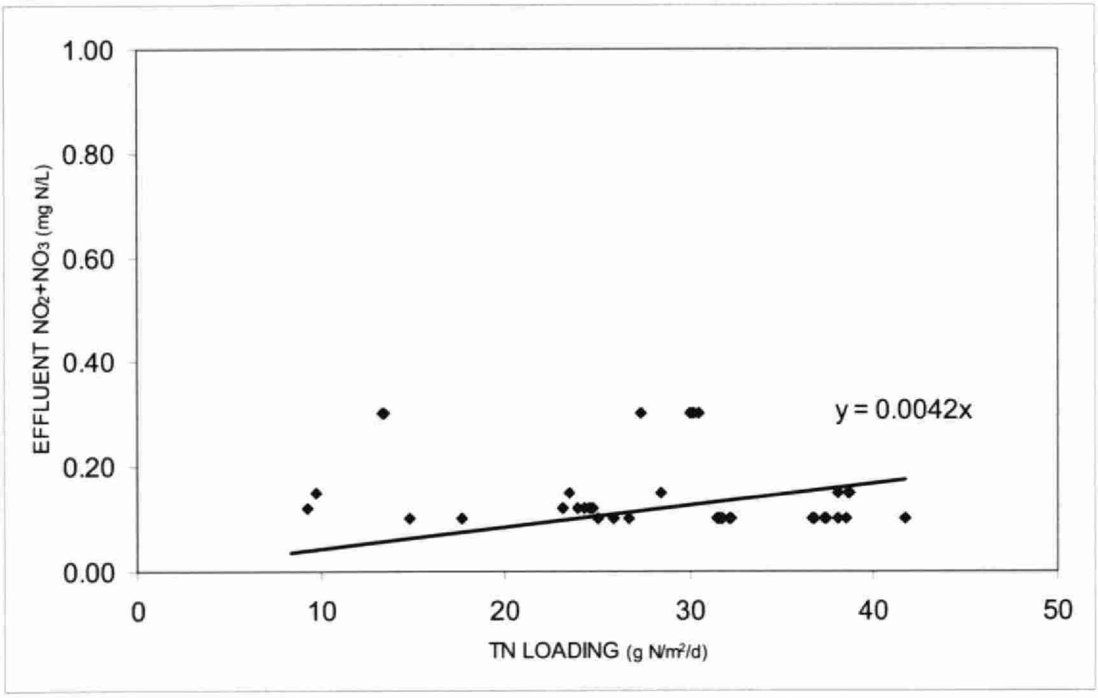

Figure A.V 2.5: RSF #2 Acclimation Phase TN Loading vs. Effluent NO_2 + NO_3

A.V 2.2 RSF #2 Phase 1

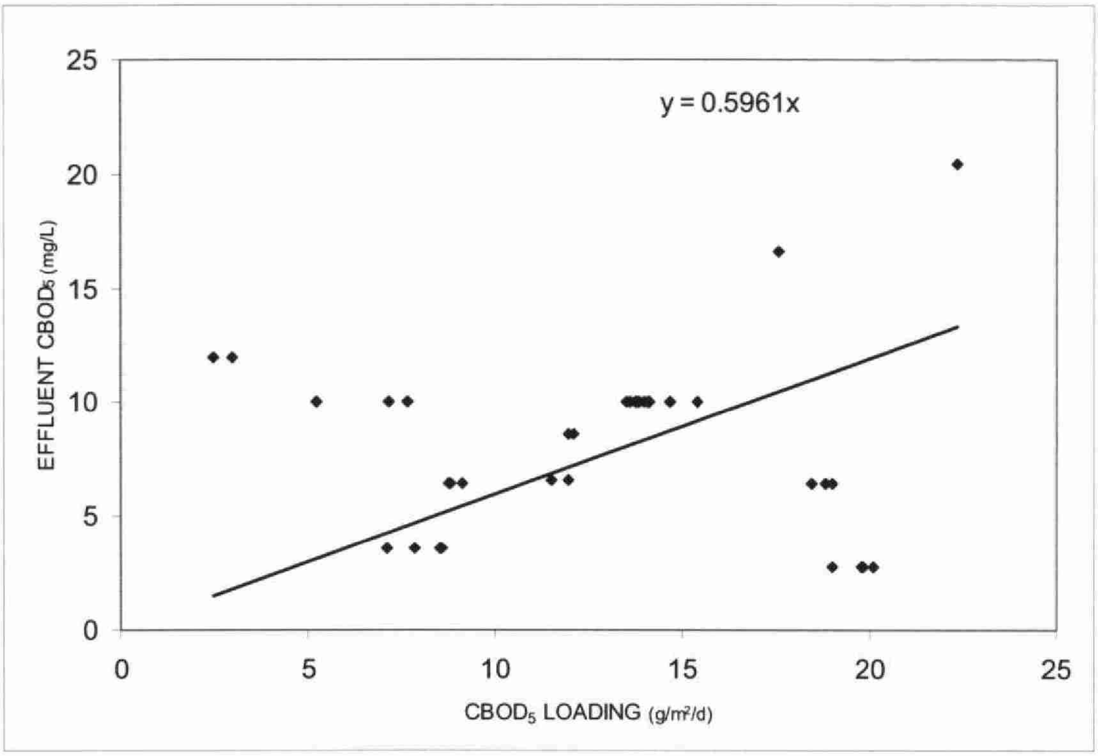

Figure A.V 2.6: RSF #2 Phase 1 $CBOD_5$ Loading vs. Effluent $CBOD_5$

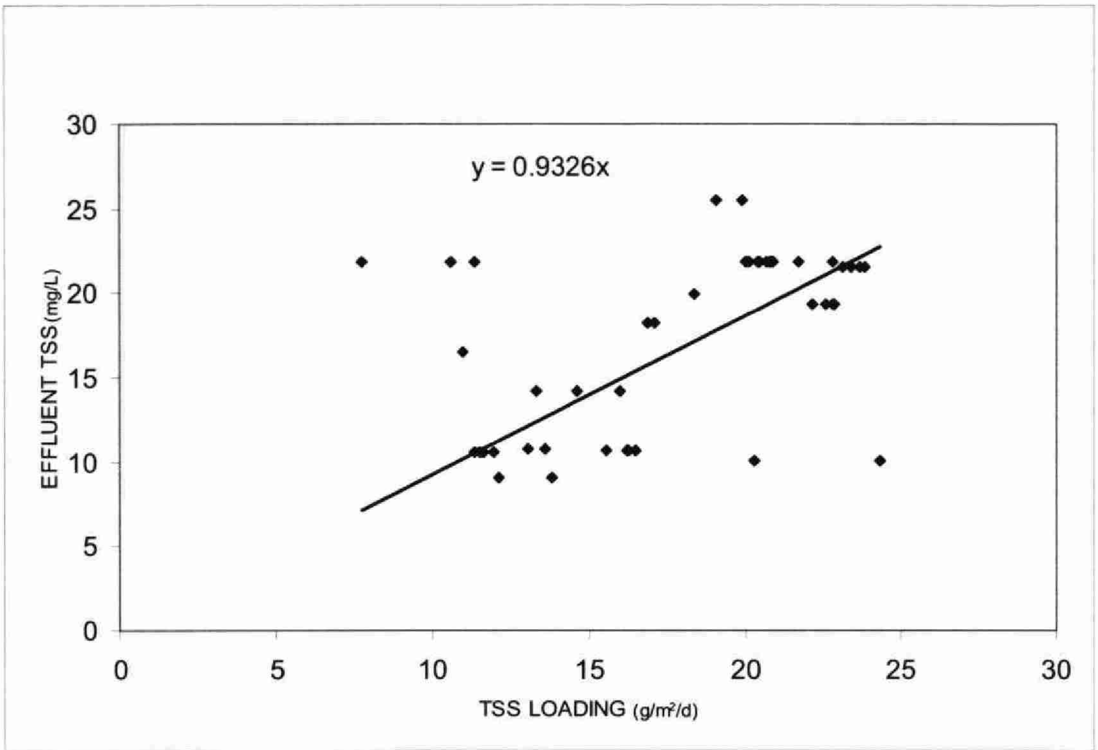

Figure A.V 2.7: RSF #2 Phase 1 TSS Loading vs. Effluent TSS

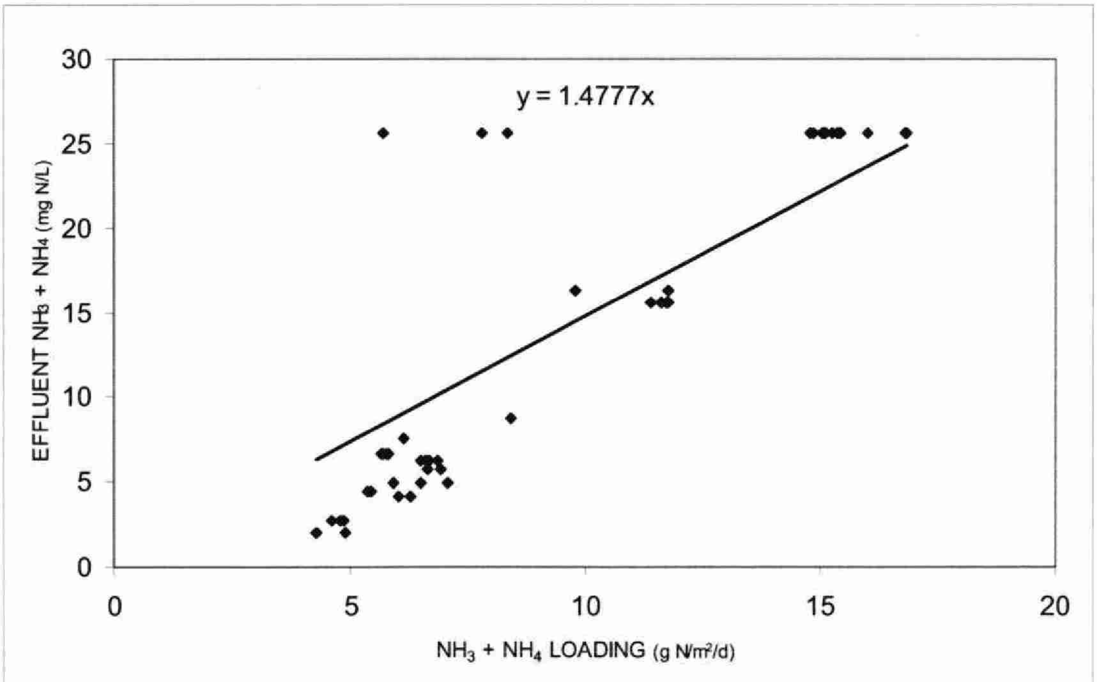

Figure A.V 2.8: RSF #2 Phase 1 NH_3 +NH_4 Loading vs. Effluent NH_3 +NH_4

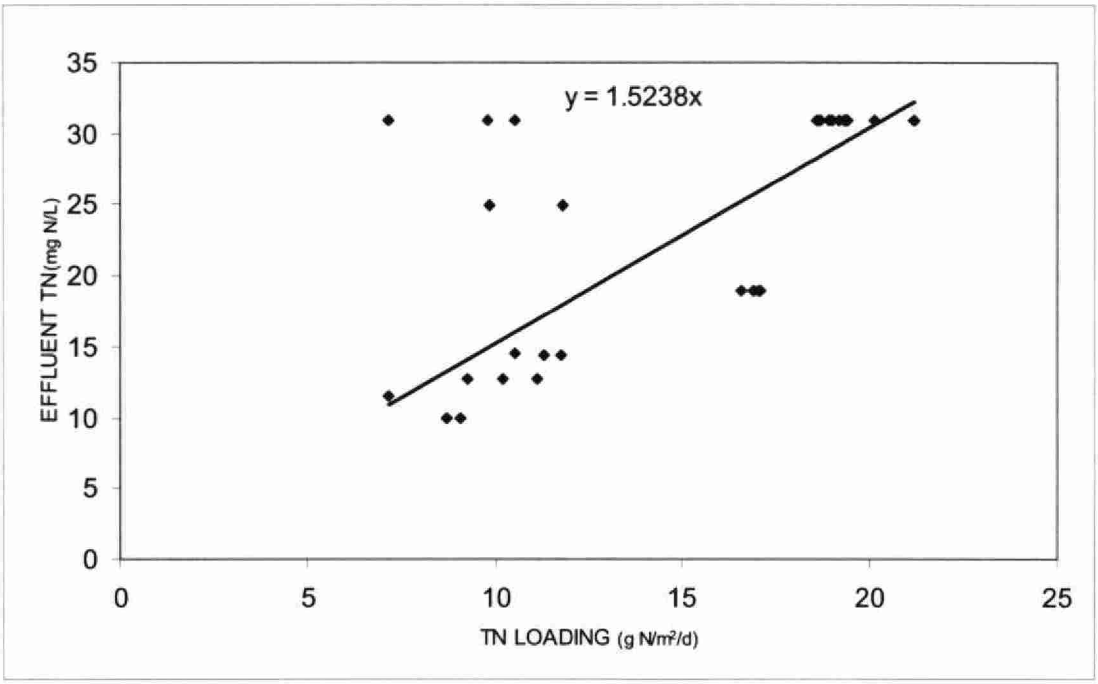

Figure A.V 2.9: RSF #2 Phase 1 TN Loading vs. Effluent TN

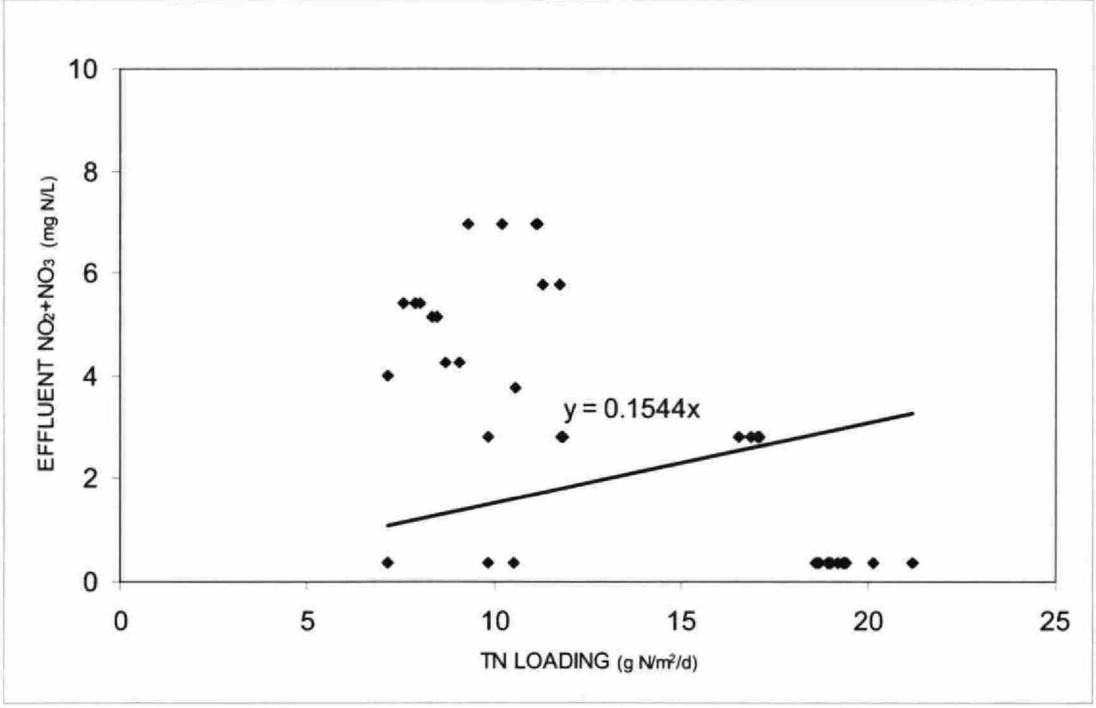

Figure A.V 2.10: RSF #2 Phase 1 TN Loading vs. Effluent $NO_2 + NO_3$

A.V 2.3 RSF #2 Phase 2

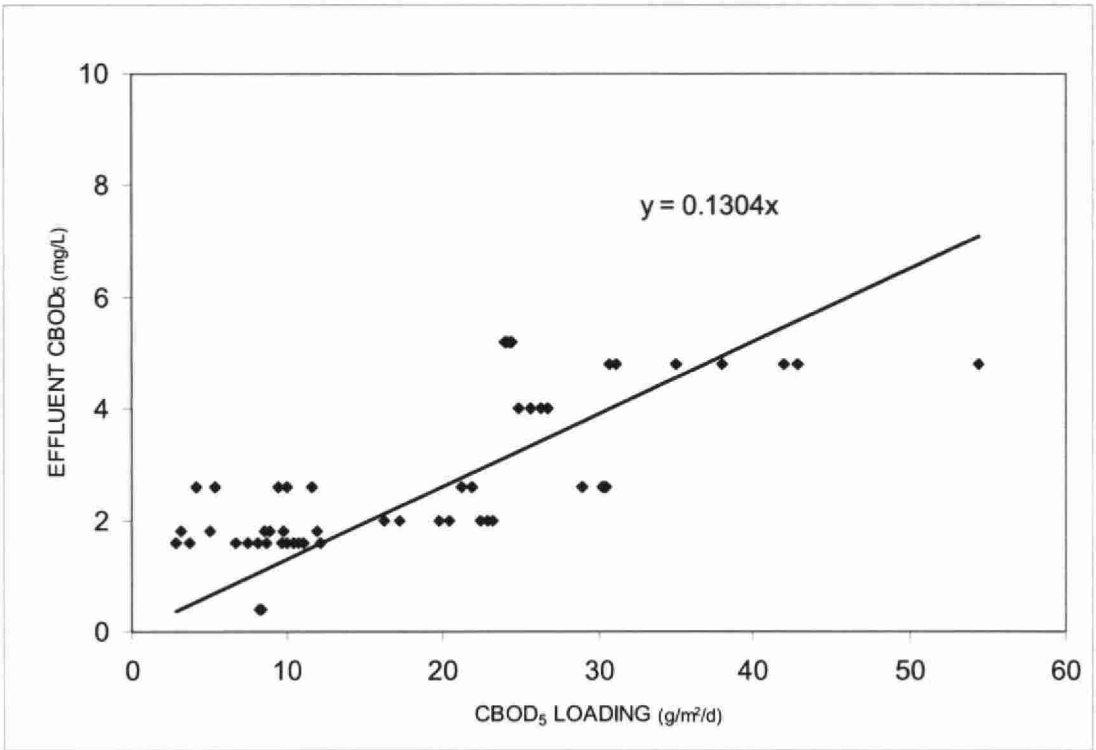

Figure A.V 2.11: RSF #2 Phase 2 $CBOD_5$ Loading vs. Effluent $CBOD_5$

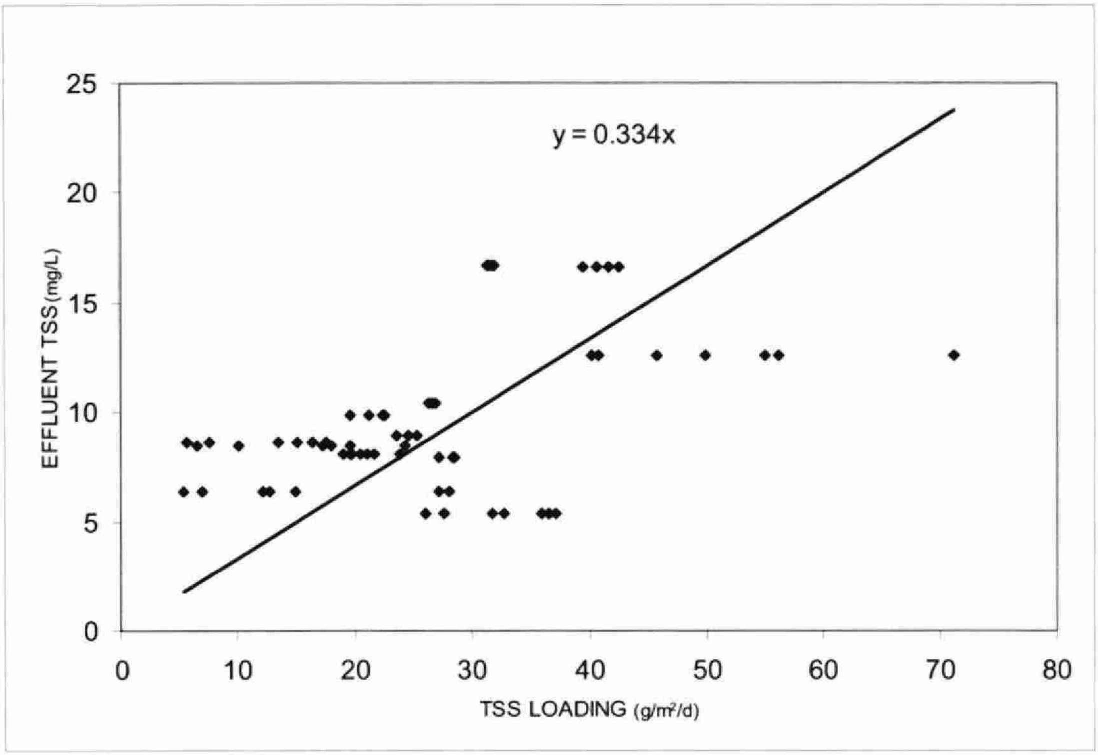

Figure A.V 2.12: RSF #2 Phase 2 TSS Loading vs. Effluent TSS

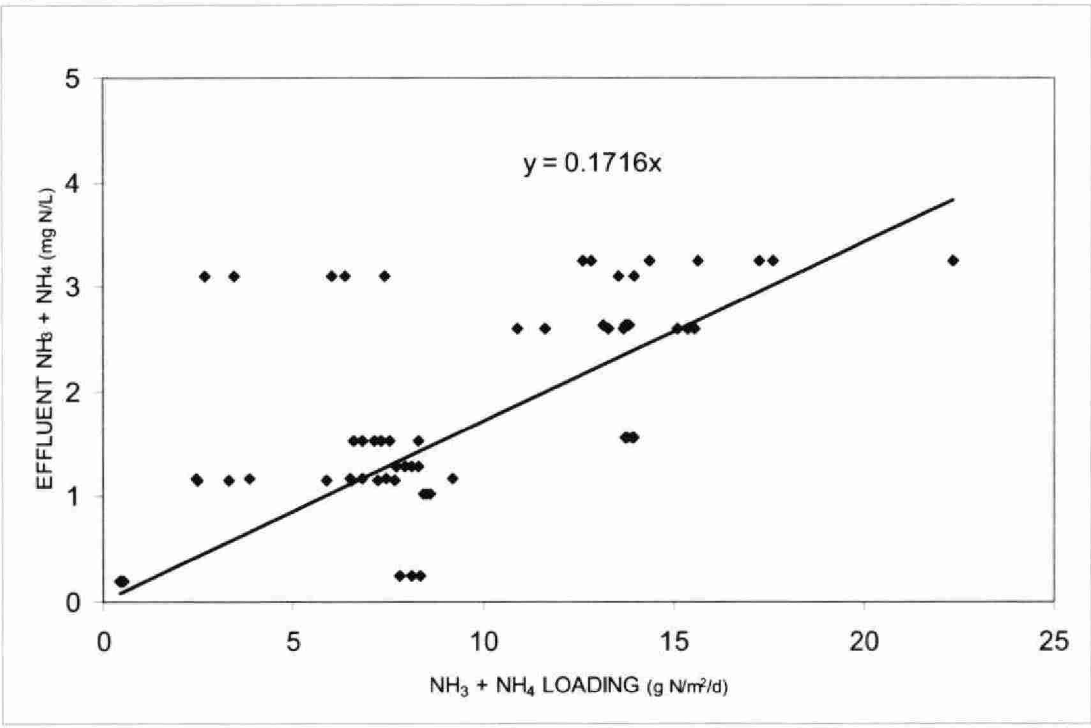

Figure A.V 2.13:RSF #2 Phase 2 NH_3 +NH_4 Loading vs. Effluent NH_3 +NH_4

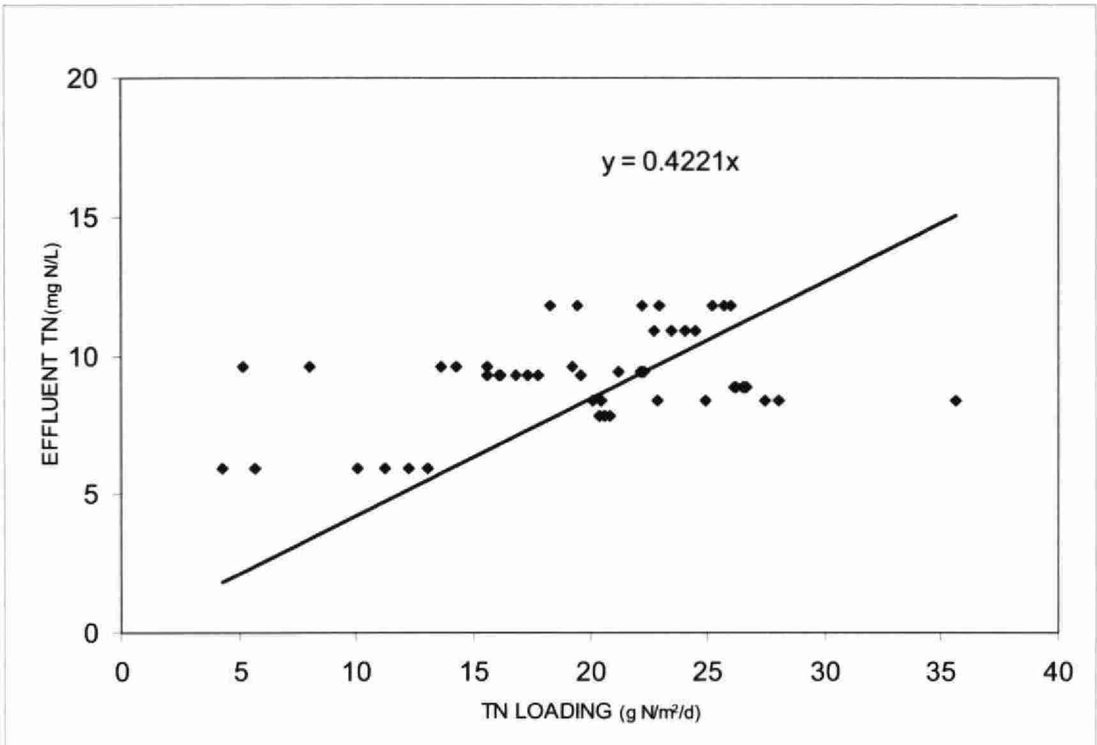

Figure A.V 2.14: RSF #2 Phase 2 TN Loading vs. Effluent TN

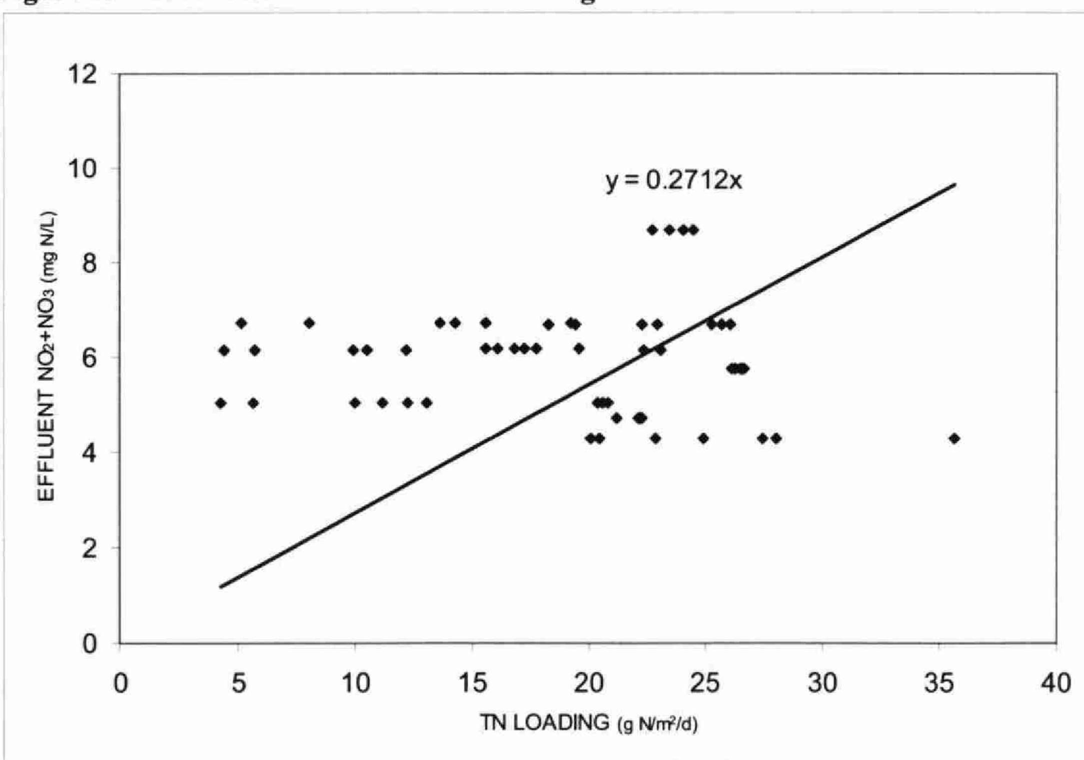

Figure A.V 2.15: RSF #2 Phase 2 TN Loading vs. Effluent $NO_2 + NO_3$

A.V 2.4 RSF #2 Phase 3

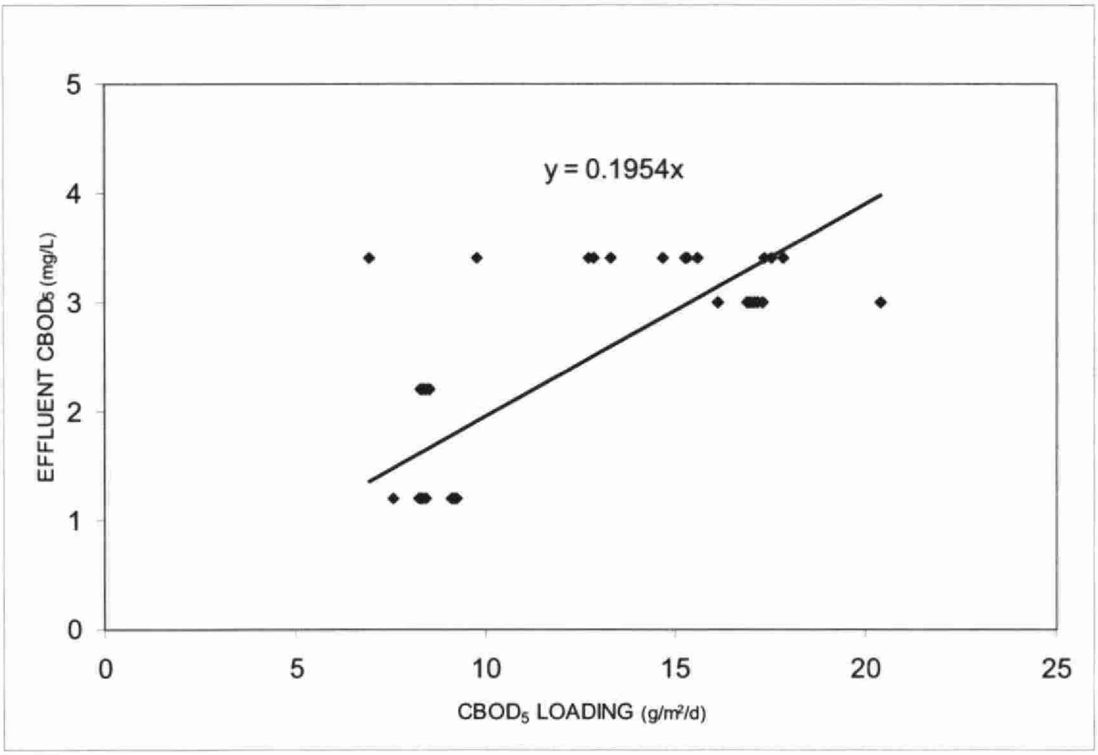

Figure A.V 2.16: RSF #2 Phase 3 $CBOD_5$ Loading vs. Effluent $CBOD_5$

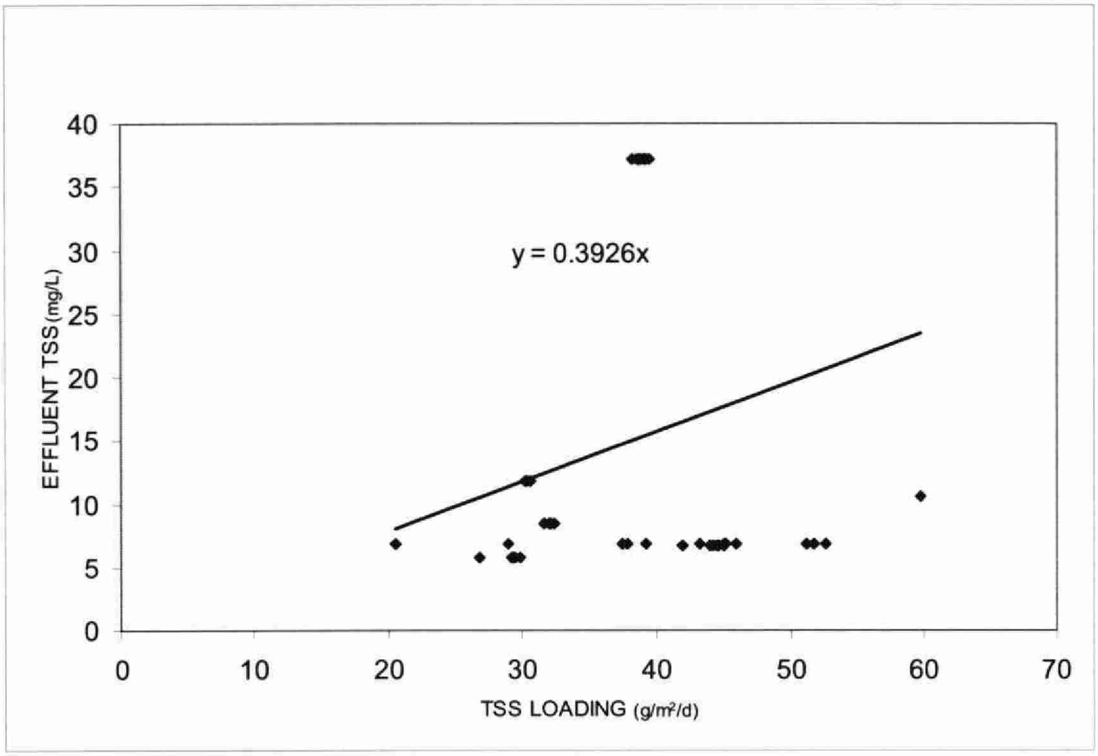

Figure A.V 2.17: RSF #2 Phase 3 TSS Loading vs. Effluent TSS

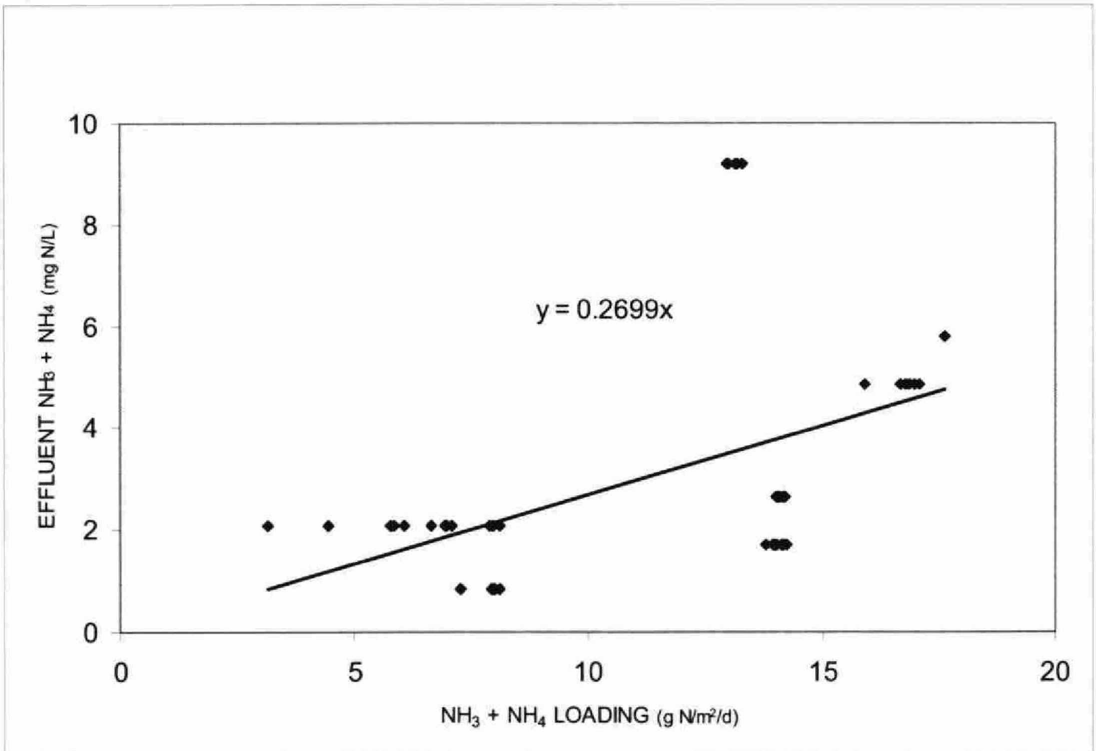

Figure A.V 2.18:RSF #2 Phase 3 NH_3 +NH_4 Loading vs. Effluent NH_3 +NH_4

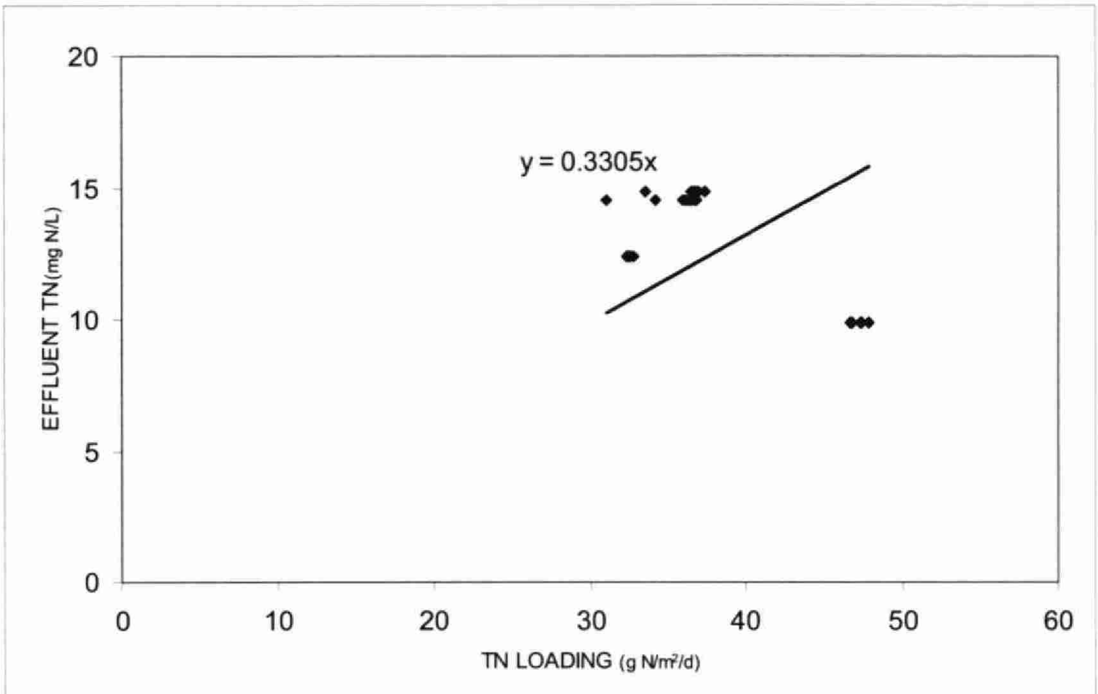

Figure A.V 2.19: RSF #2 Phase 3 TN Loading vs. Effluent TN

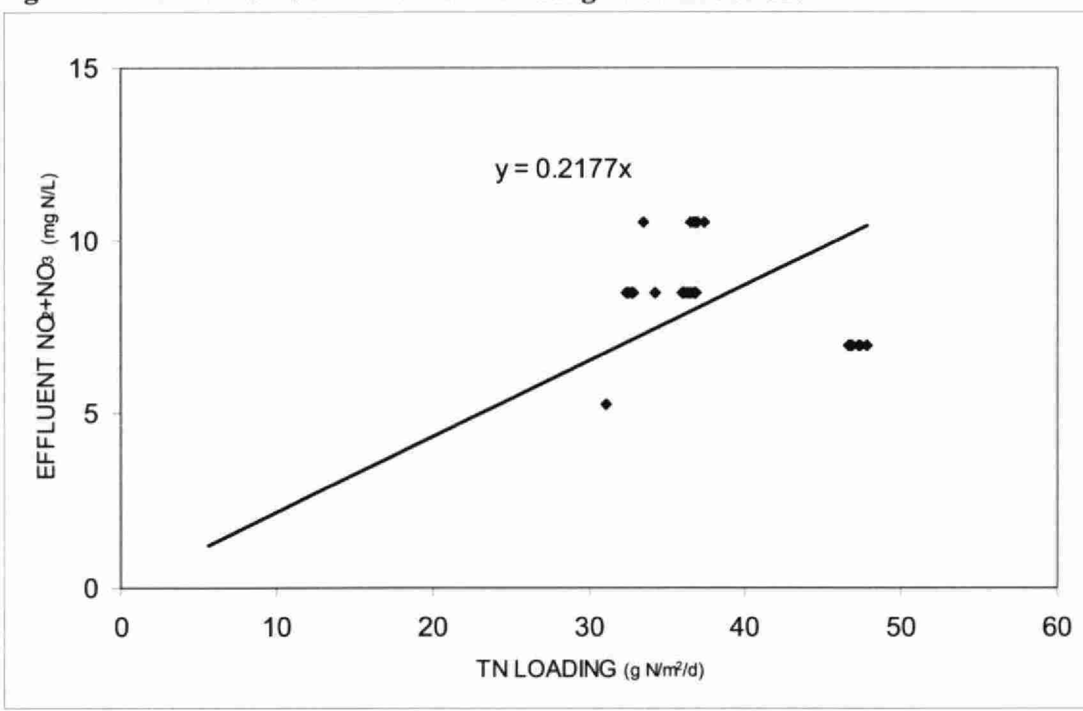

Figure A.V 2.20: RSF #2 Phase 3 TN Loading vs. Effluent NO_2 + NO_3

AV 2.5 RSF #2 Phase 4

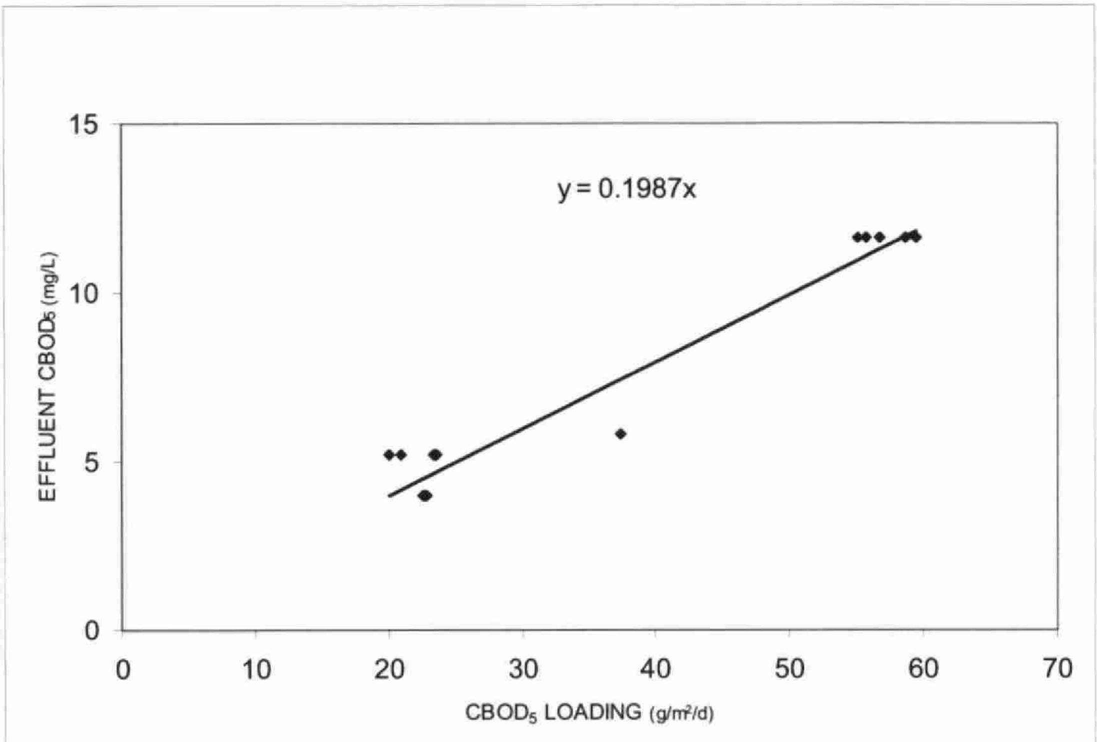

Figure A.V 2.21: RSF #2 Phase 4 $CBOD_5$ Loading vs. Effluent $CBOD_5$

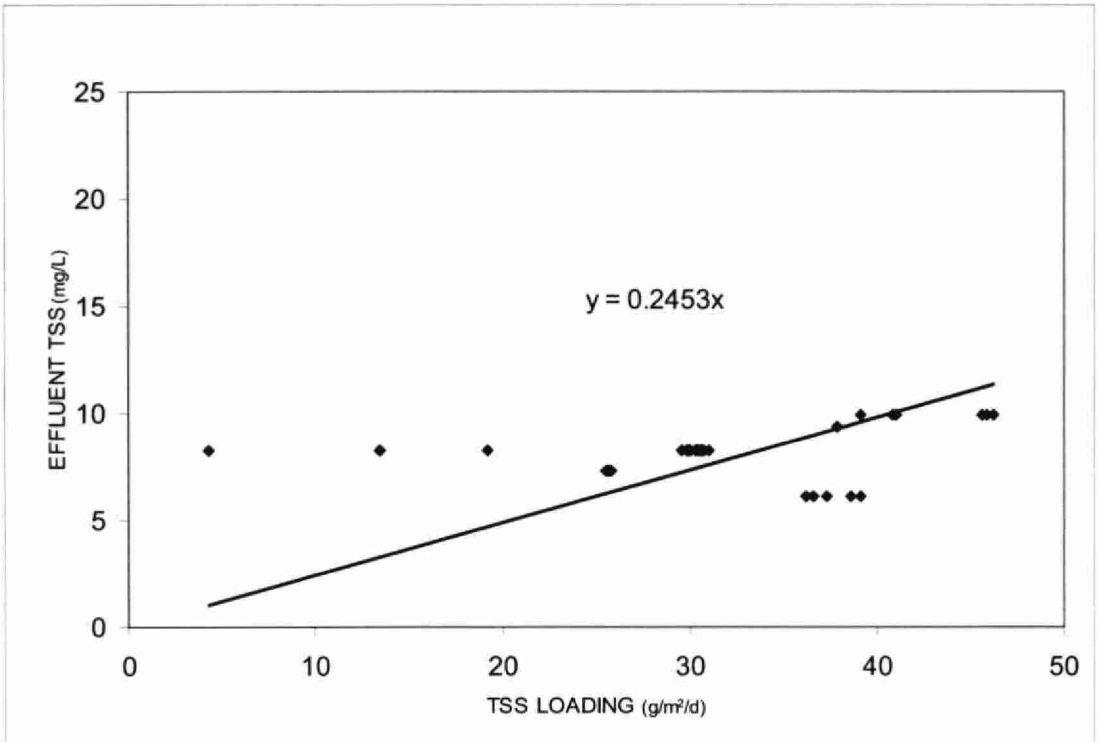

Figure A.V 2.22: RSF #2 Phase 4 TSS Loading vs. Effluent TSS

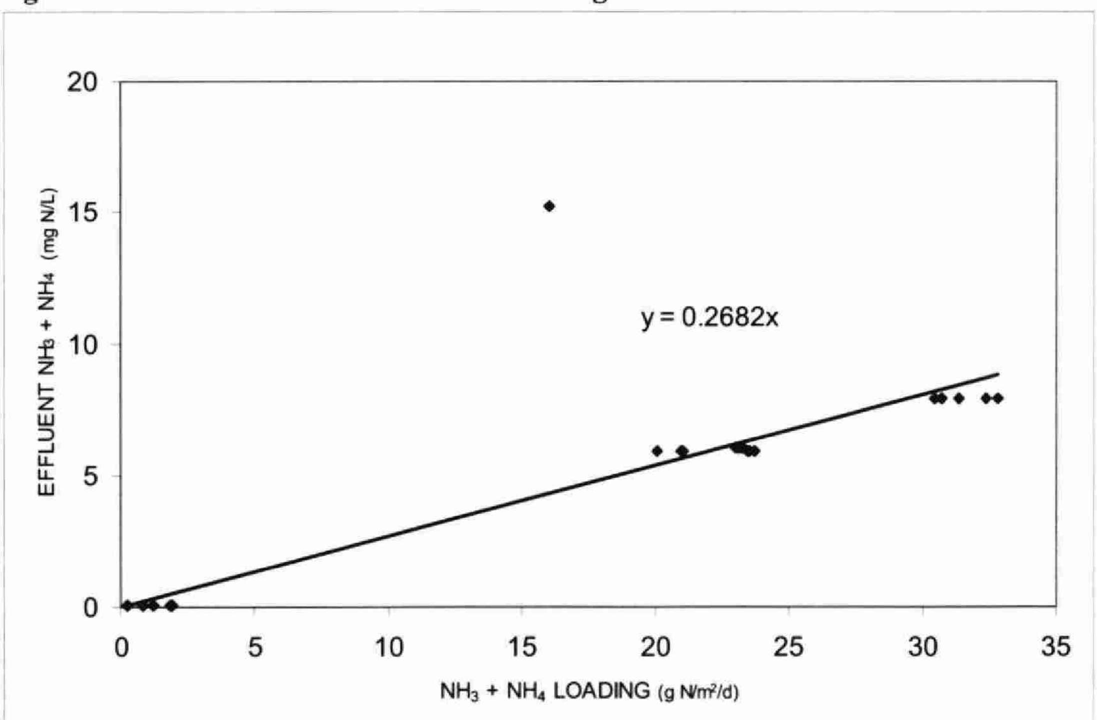

Figure A.V 2.23:RSF #2 Phase 4 NH_3 +NH_4 Loading vs. Effluent NH_3 +NH_4

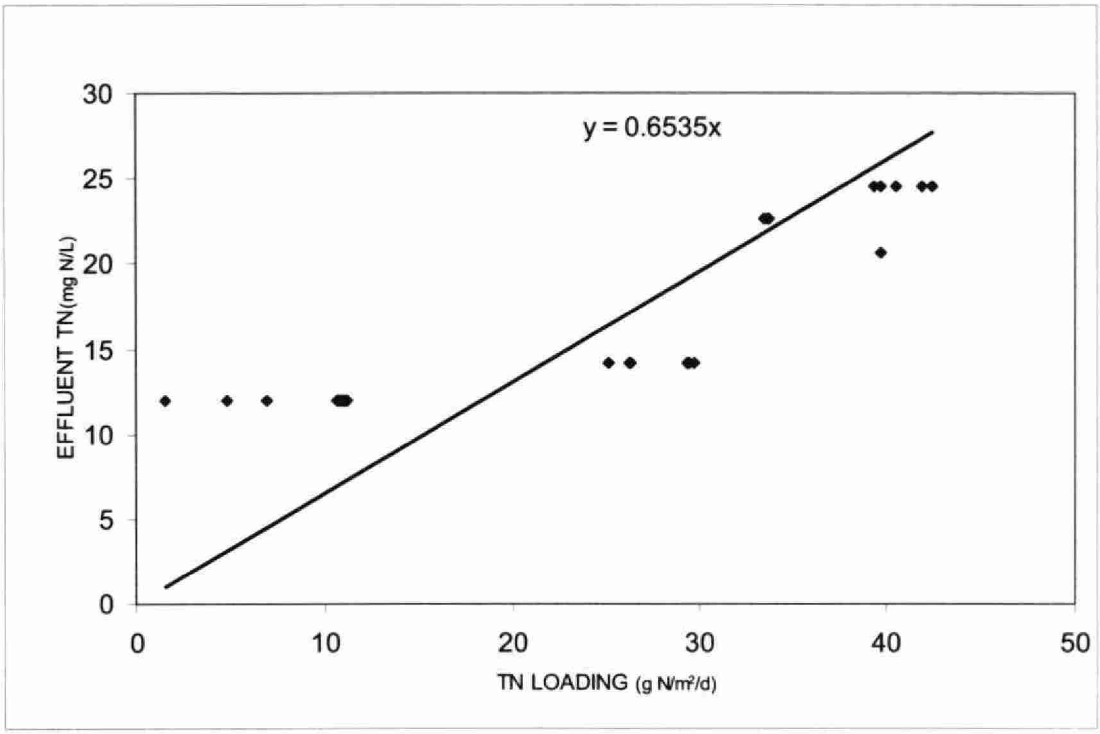

Figure A.V 2.24: RSF #2 Phase 4 TN Loading vs. Effluent TN

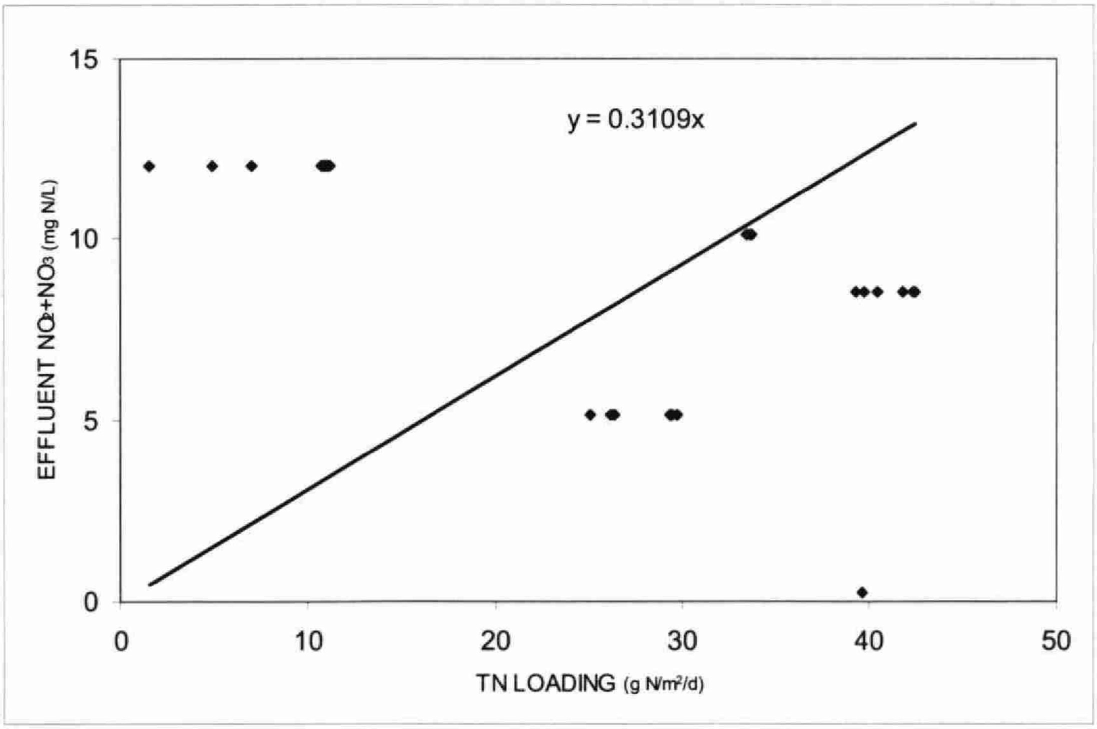

Figure A.V 2.25: RSF #2 Phase 4 TN Loading vs. Effluent $NO_2 + NO_3$

A.V 3.0 RSF #3 Loadings

A.V 3.1 RSF #3 Acclimation Phase

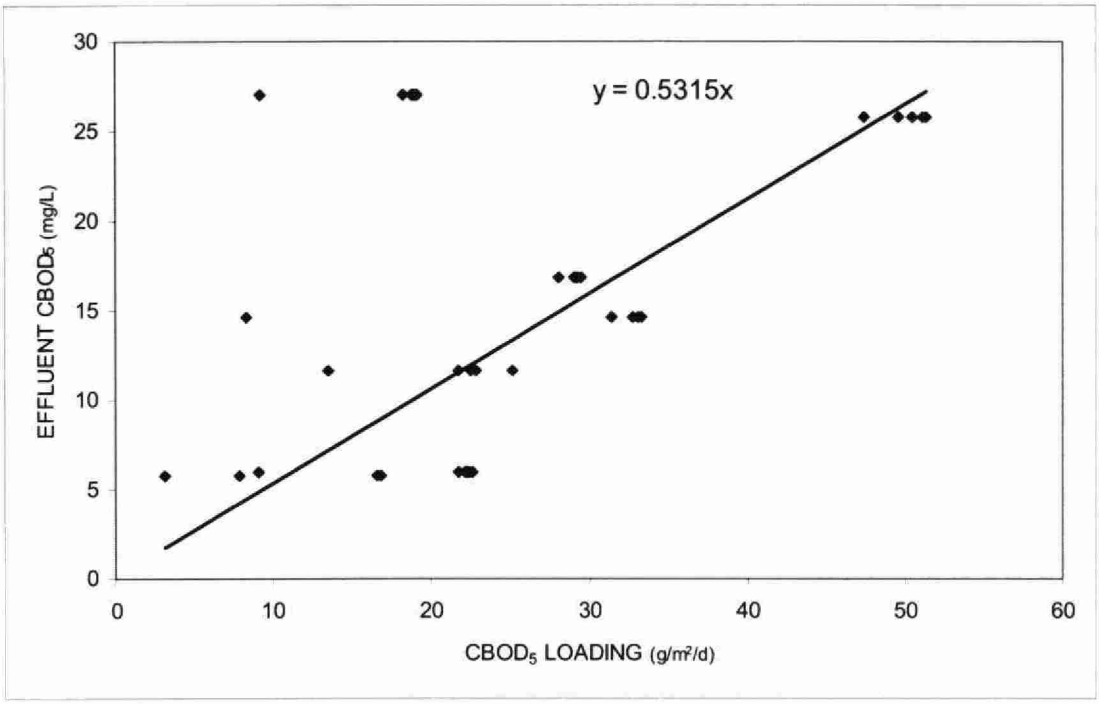

Figure A.V 3.1: RSF #3 Acclimation Phase $CBOD_5$ Loading vs. Effluent $CBOD_5$

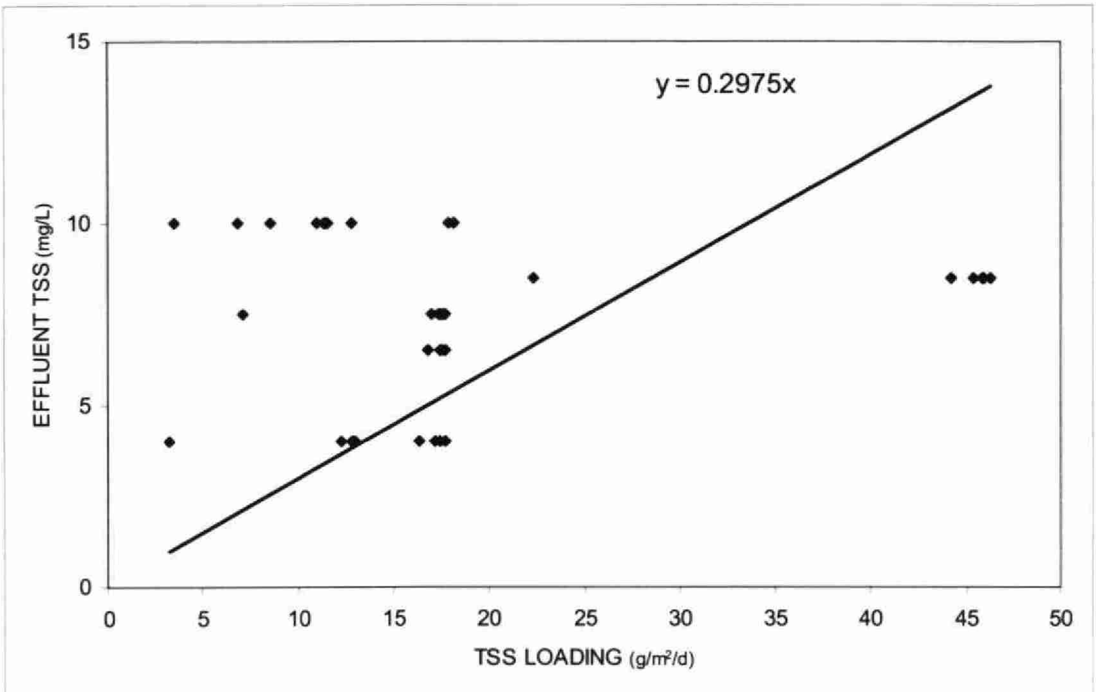

Figure A.V 3.2: RSF #3 Acclimation Phase TSS Loading vs. Effluent TSS

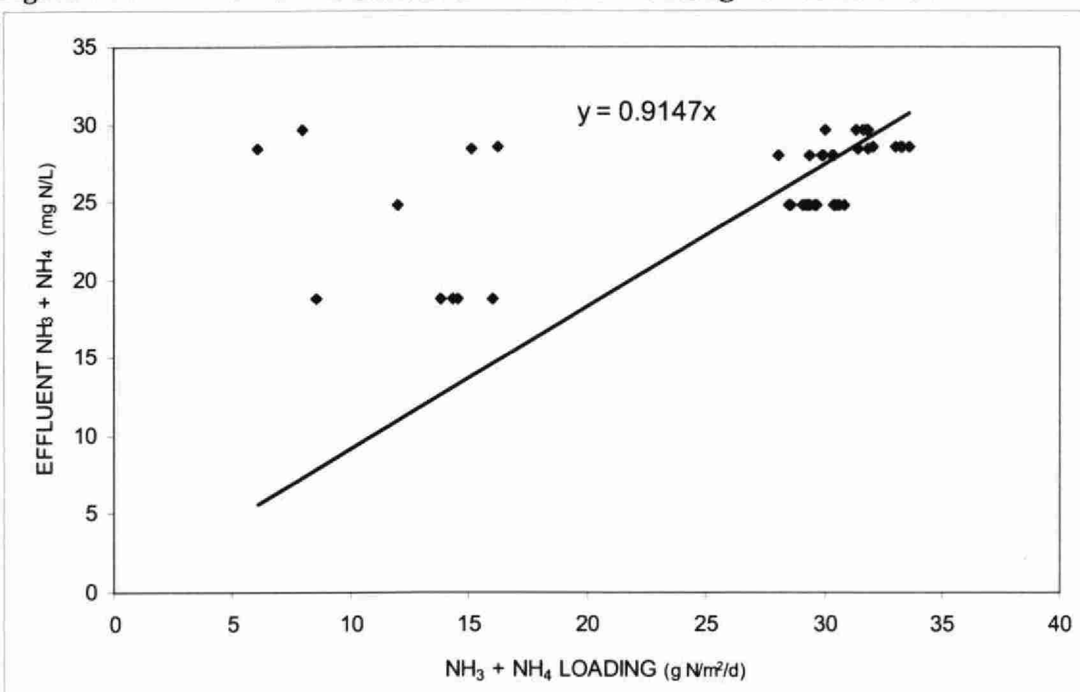

Figure A.V 3.3:RSF #3 Acclimation Phase NH_3 +NH_4 Loading vs. Effluent NH_3 +NH_4

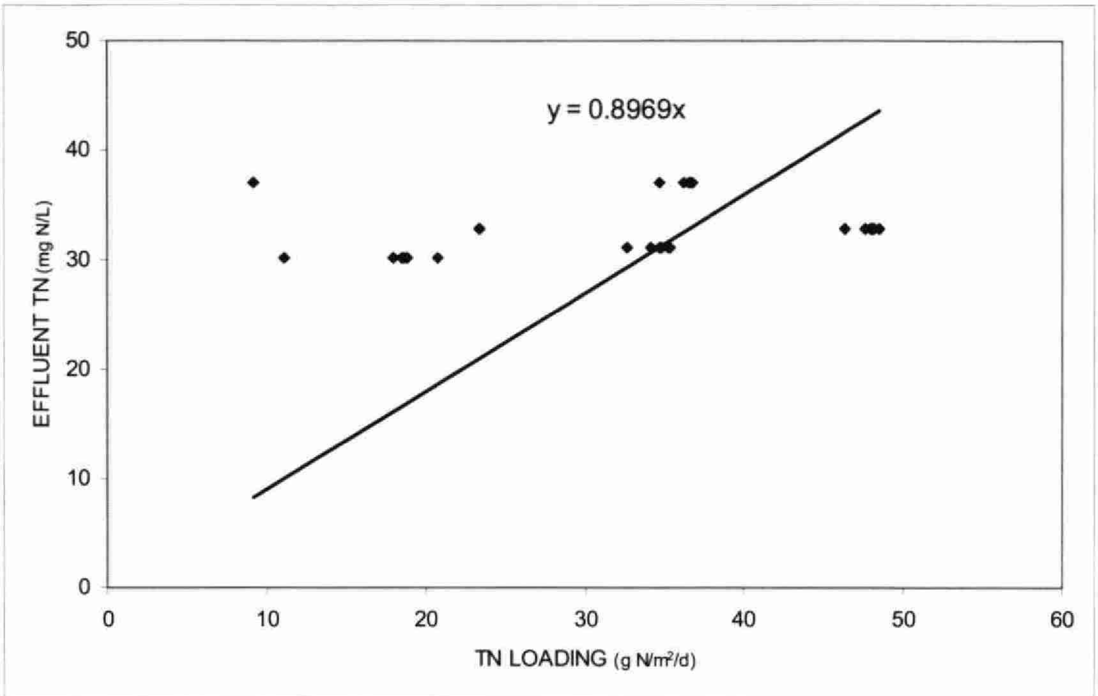

Figure A.V 3.4: RSF #3 Acclimation Phase TN Loading vs. Effluent TN

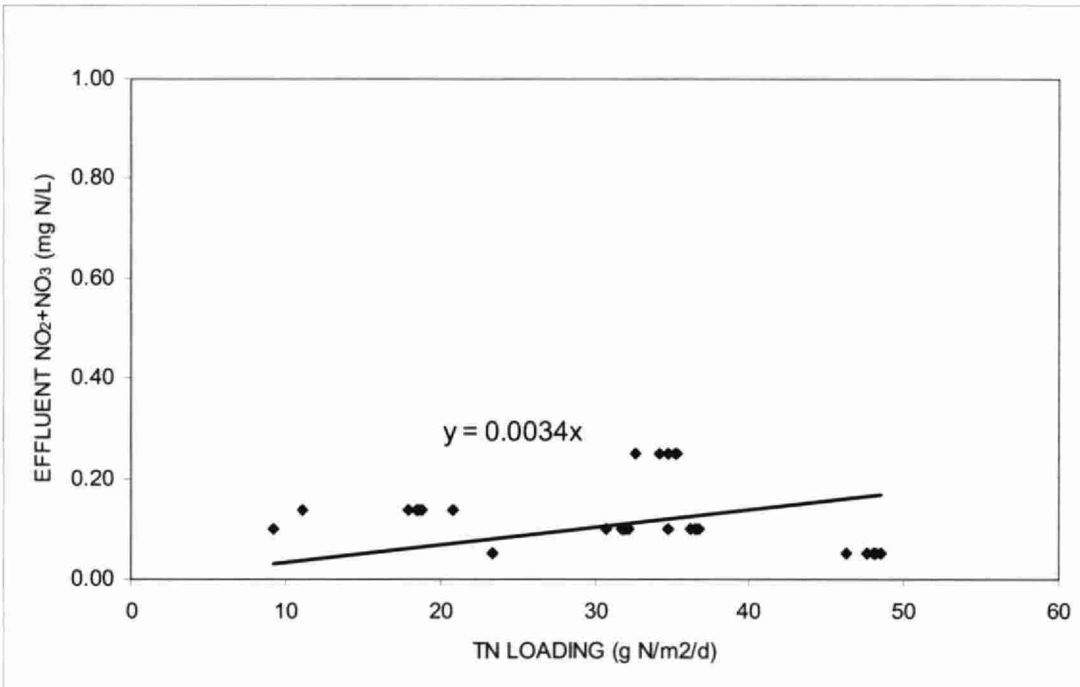

Figure A.V 3.5: RSF #3 Acclimation Phase TN Loading vs. Effluent NO_2 + NO_3

A.V 3.2 RSF #3 Phase 1

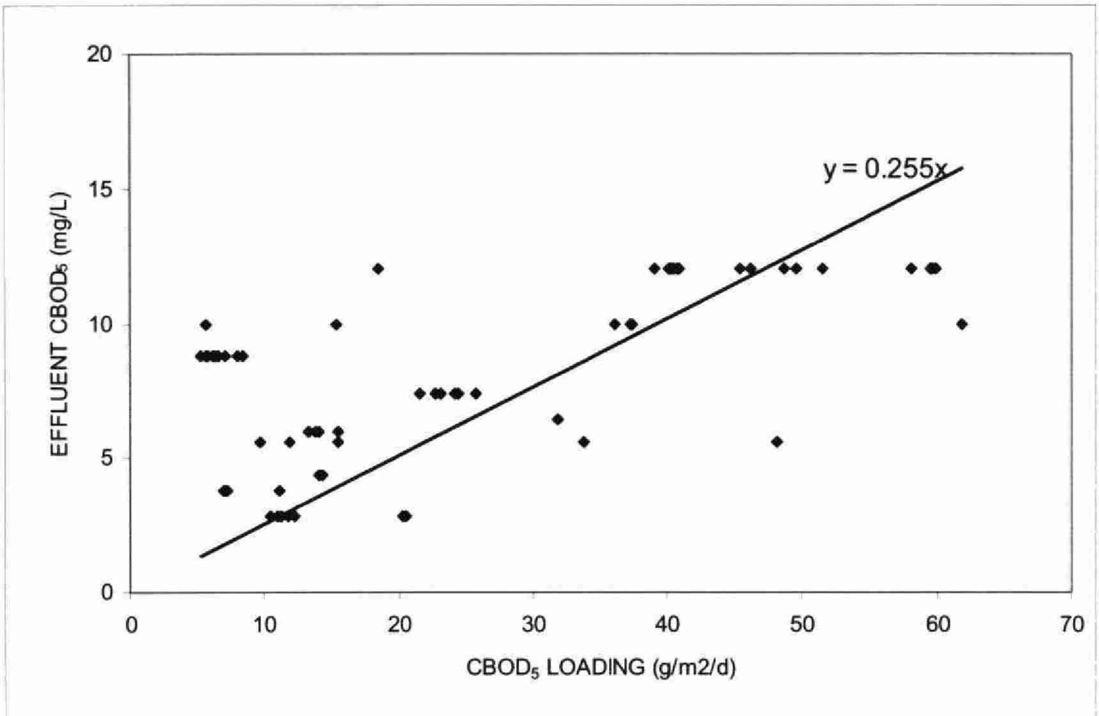

Figure A.V 3.6: RSF #3 Phase 1 $CBOD_5$ Loading vs. Effluent $CBOD_5$

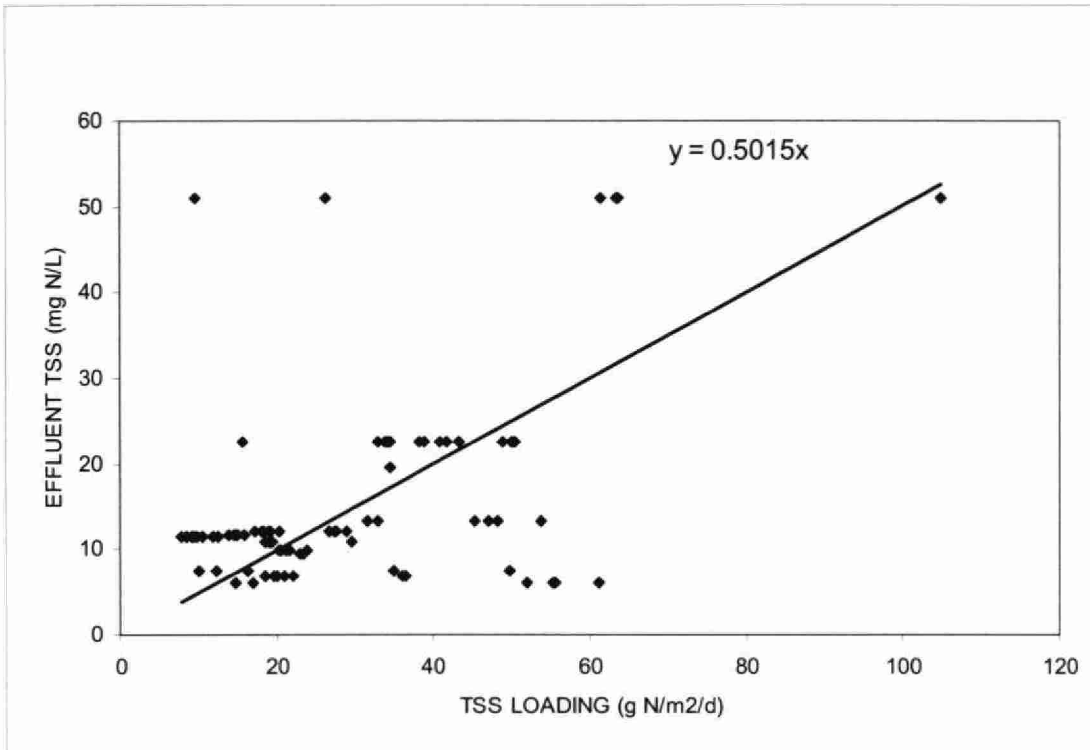

Figure A.V 3.7: RSF #3 Phase 1 TSS Loading vs. Effluent TSS

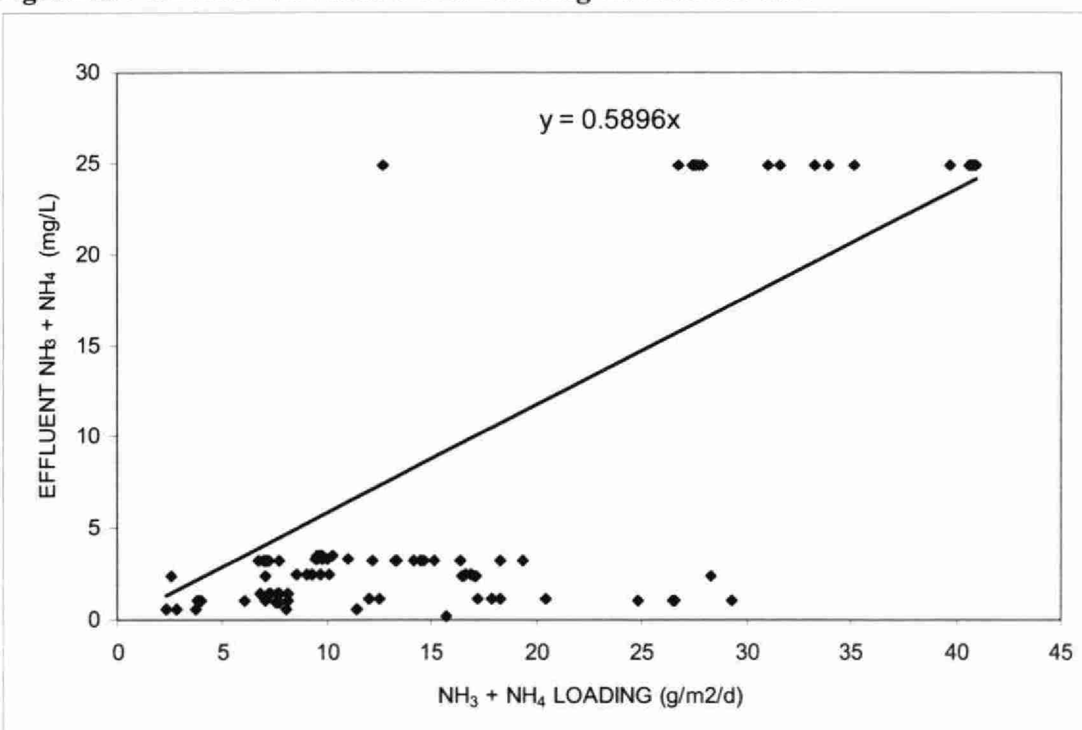

Figure A.V 3.8: RSF #3 Phase 1 NH_3 +NH_4 Loading vs. Effluent NH_3 +NH_4

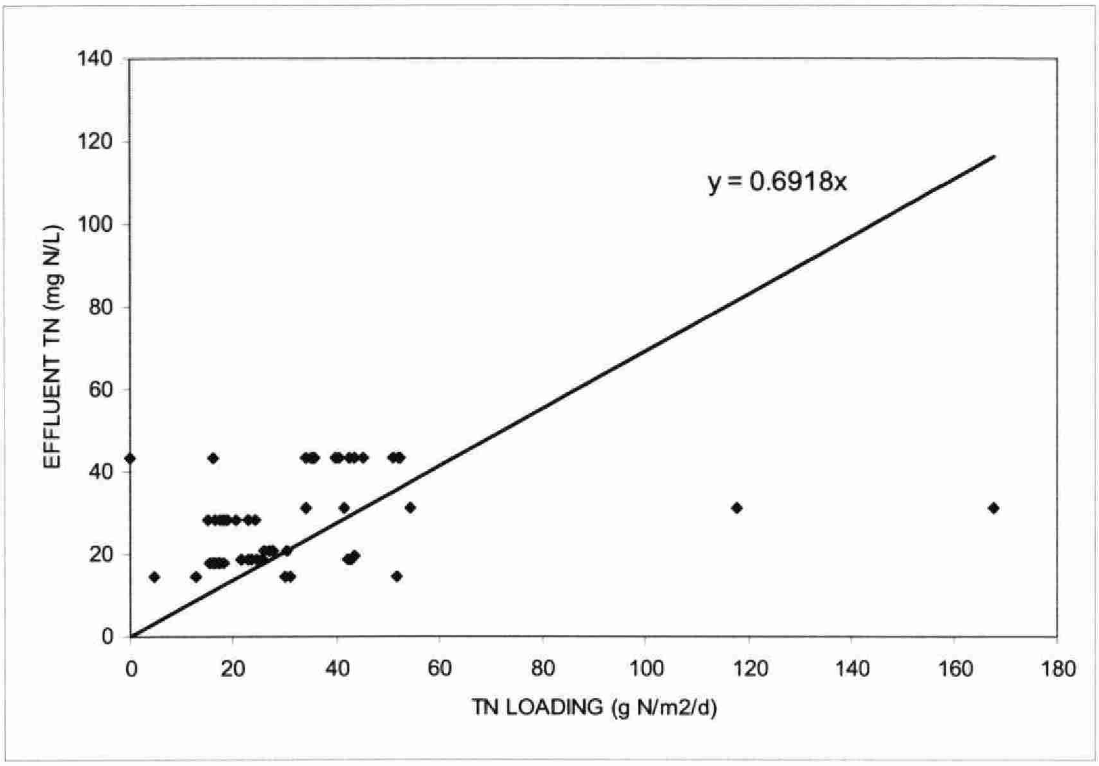

Figure A.V 3.9: RSF #3 Phase 1 TN Loading vs. Effluent TN

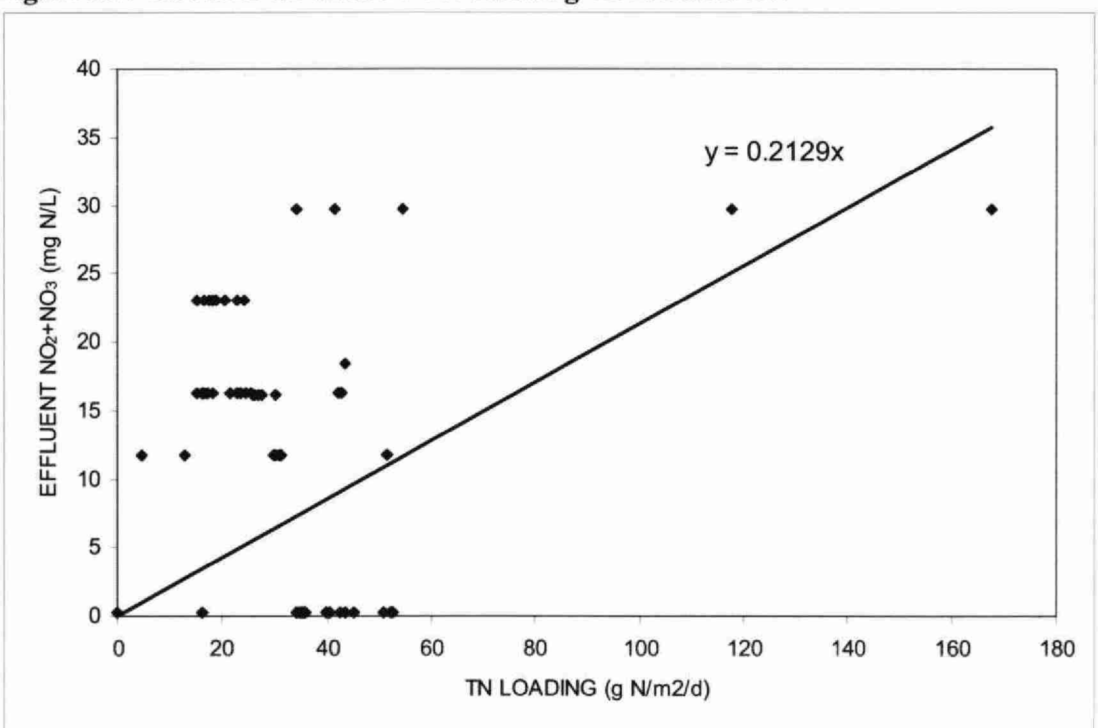

Figure A.V 3.10: RSF #3 Phase 1 TN Loading vs. Effluent NO_2 + NO_3

A.V 3.3 RSF #3 Phase 2

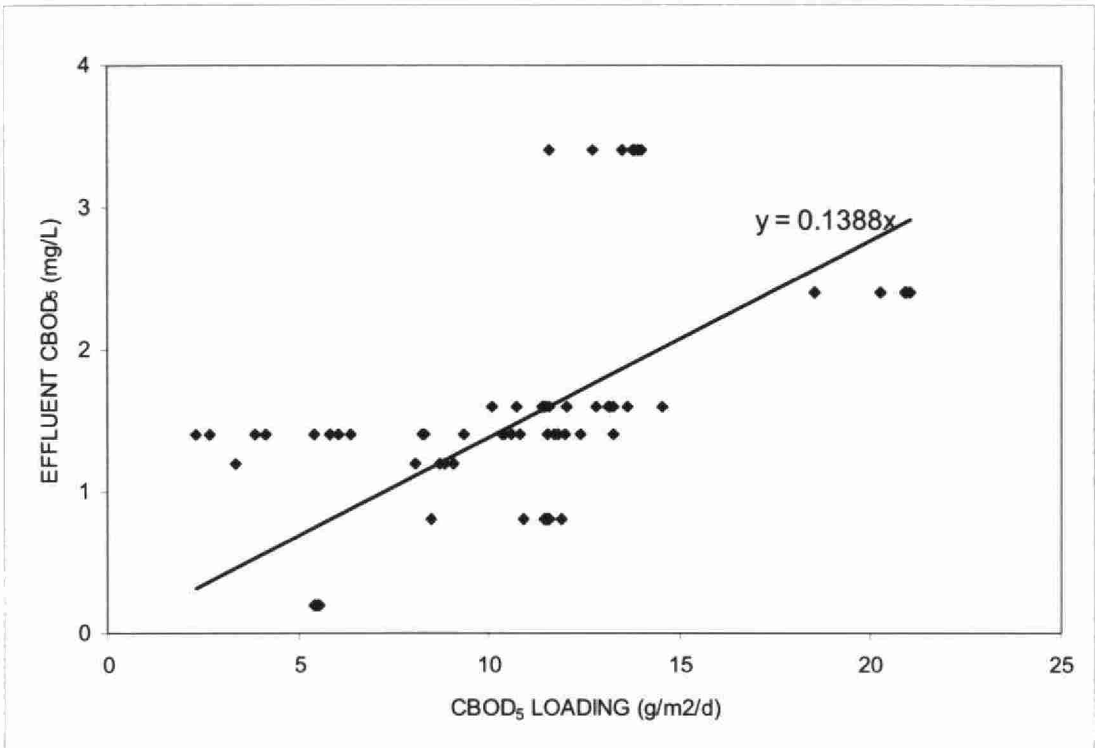

Figure A.V3.11: RSF #3 Phase 2 $CBOD_5$ Loading vs. Effluent $CBOD_5$

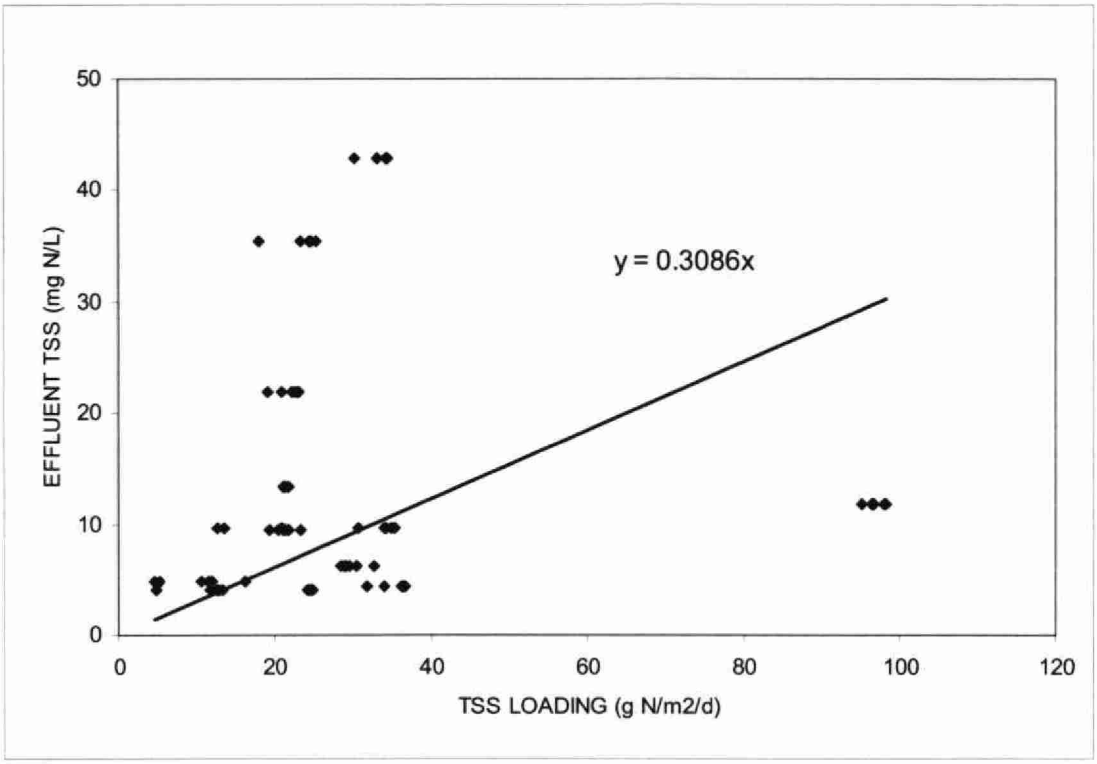

Figure A.V 3.12: RSF #3 Phase 2 TSS Loading vs. Effluent TSS

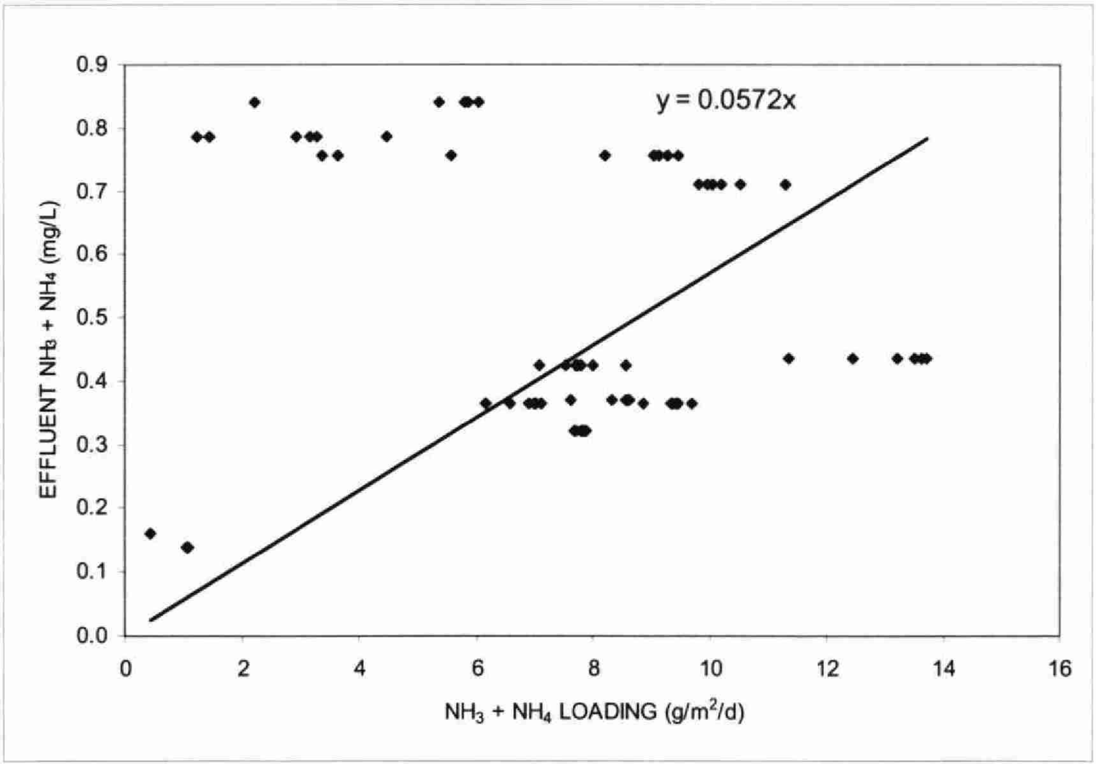

Figure A.V 3.13:RSF #3 Phase 2 NH_3 +NH_4 Loading vs. Effluent NH_3 +NH_4

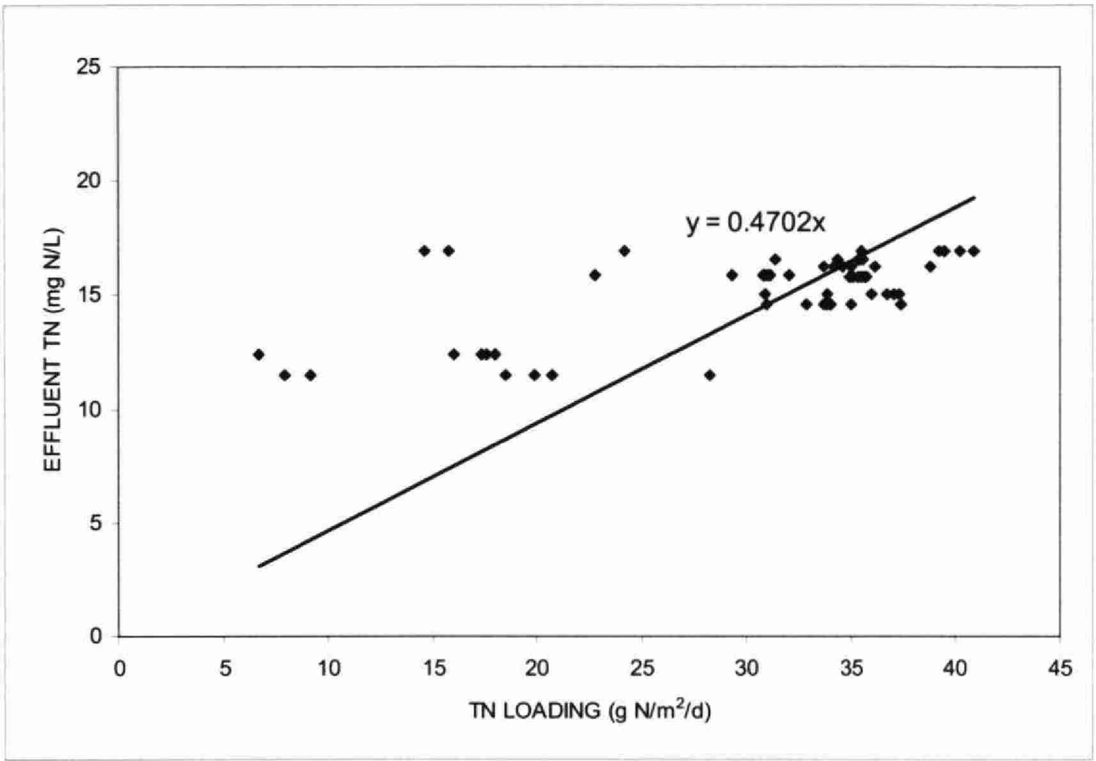

Figure A.V 3.14: RSF #3 Phase 2 TN Loading vs. Effluent TN

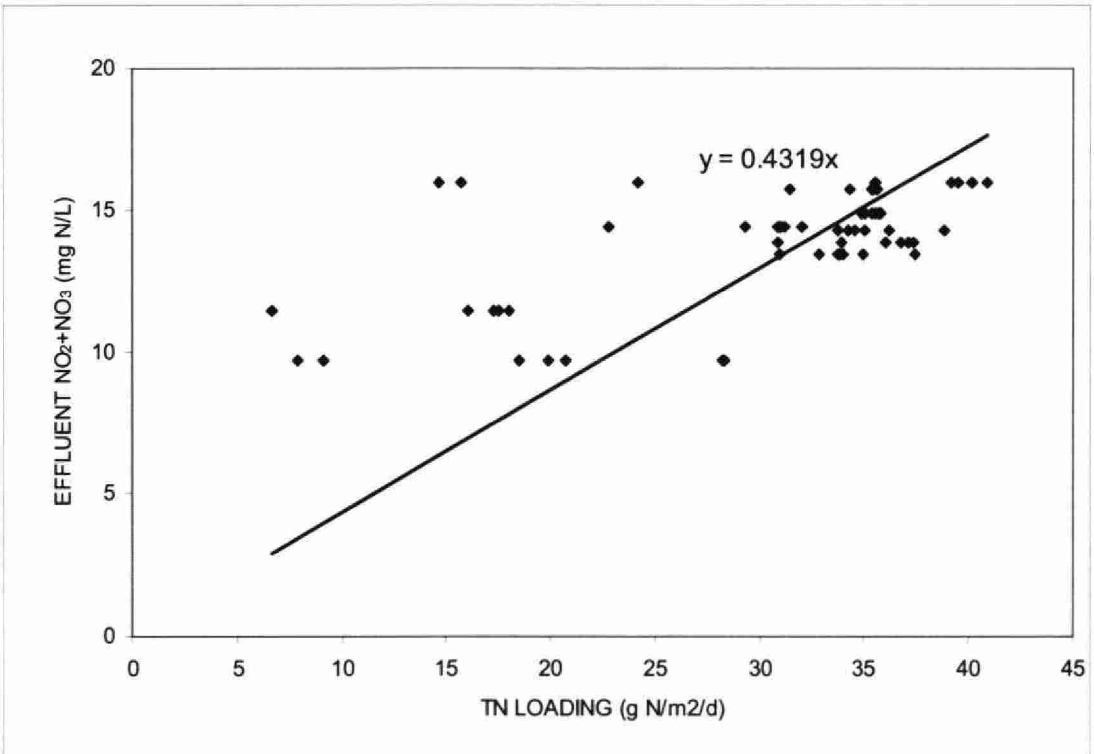

Figure A.V 3.15: RSF #3 Phase 2 TN Loading vs. Effluent $NO_2 + NO_3$

A.V 3.4 RSF #3 Phase 3

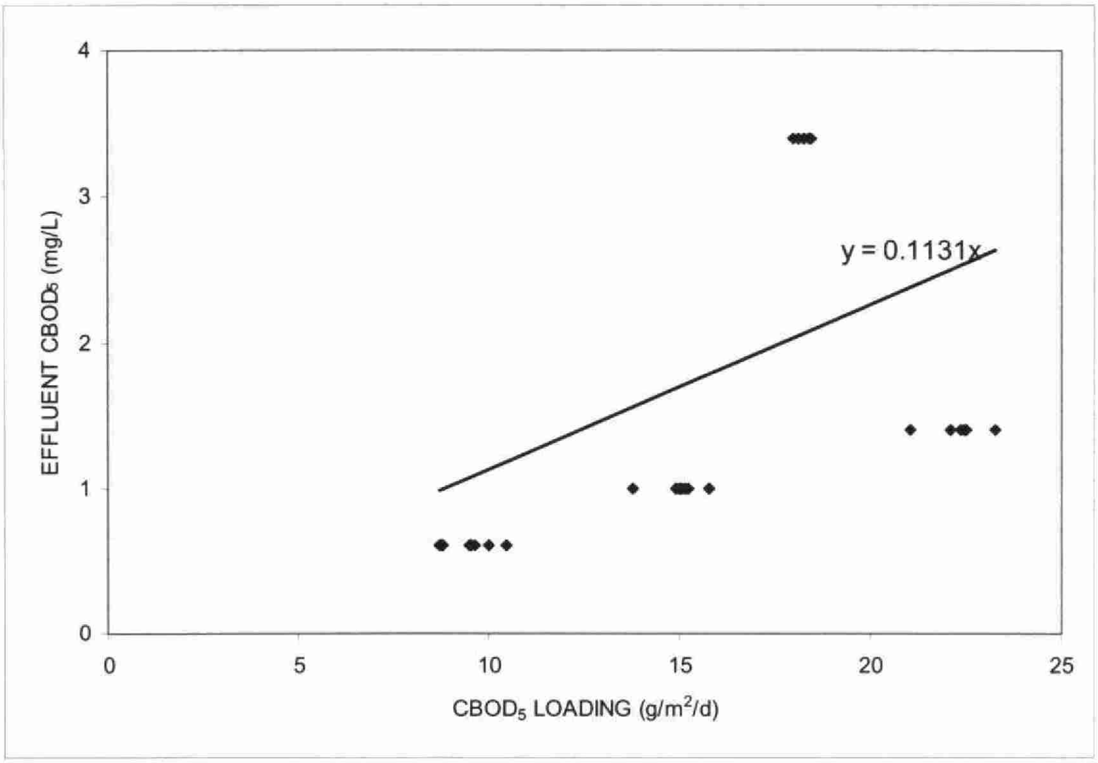

Figure A.V 3.16: RSF #3 Phase 3 $CBOD_5$ Loading vs. Effluent $CBOD_5$

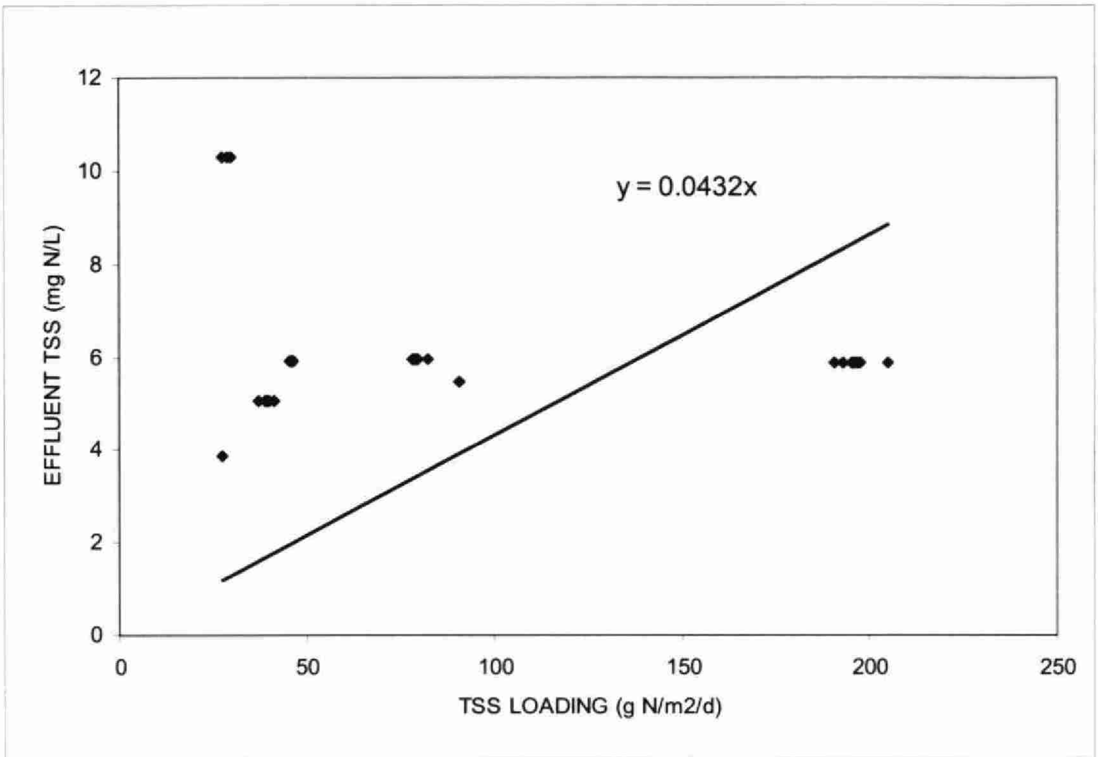

Figure A.V 3.17: RSF #3 Phase 3 TSS Loading vs. Effluent TSS

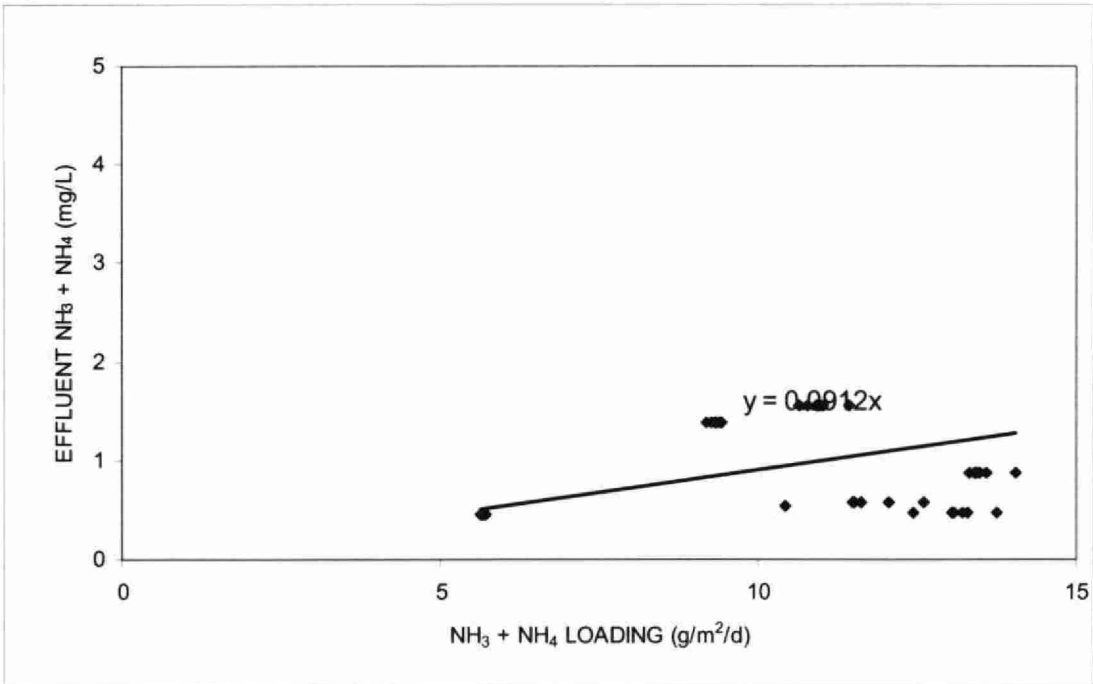

Figure A.V 3.18:RSF #3 Phase 3 NH_3 +NH_4 Loading vs. Effluent NH_3 +NH_4

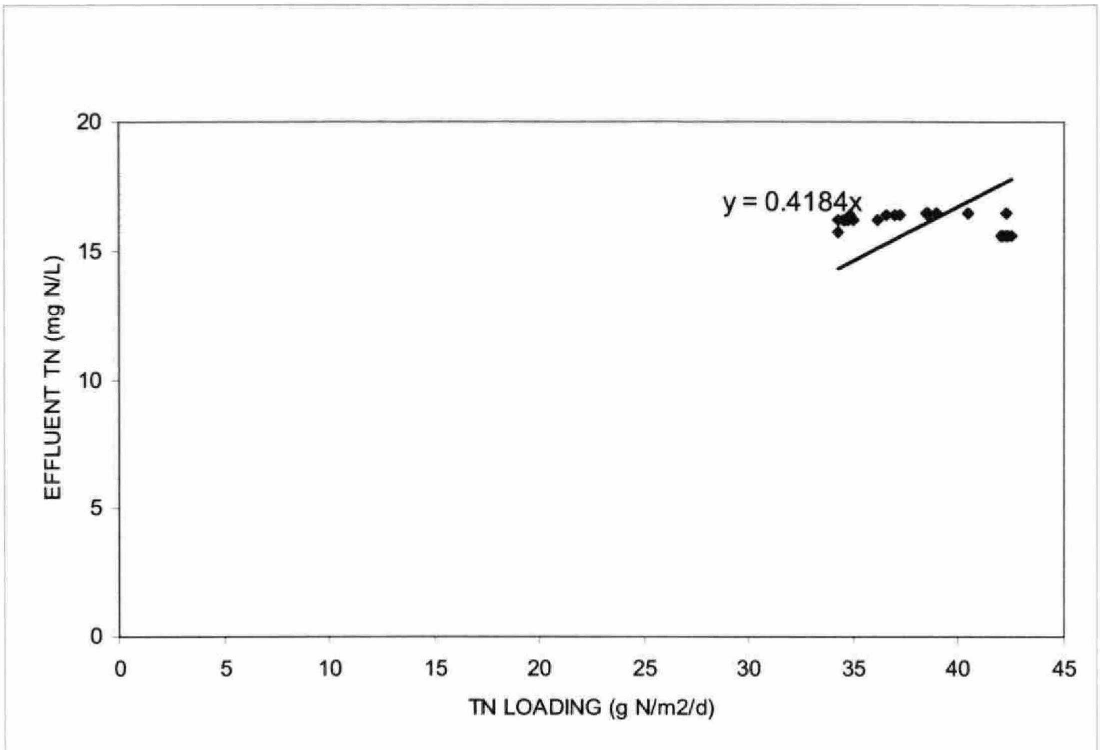

Figure A.V 3.19: RSF #3 Phase 3 TN Loading vs. Effluent TN

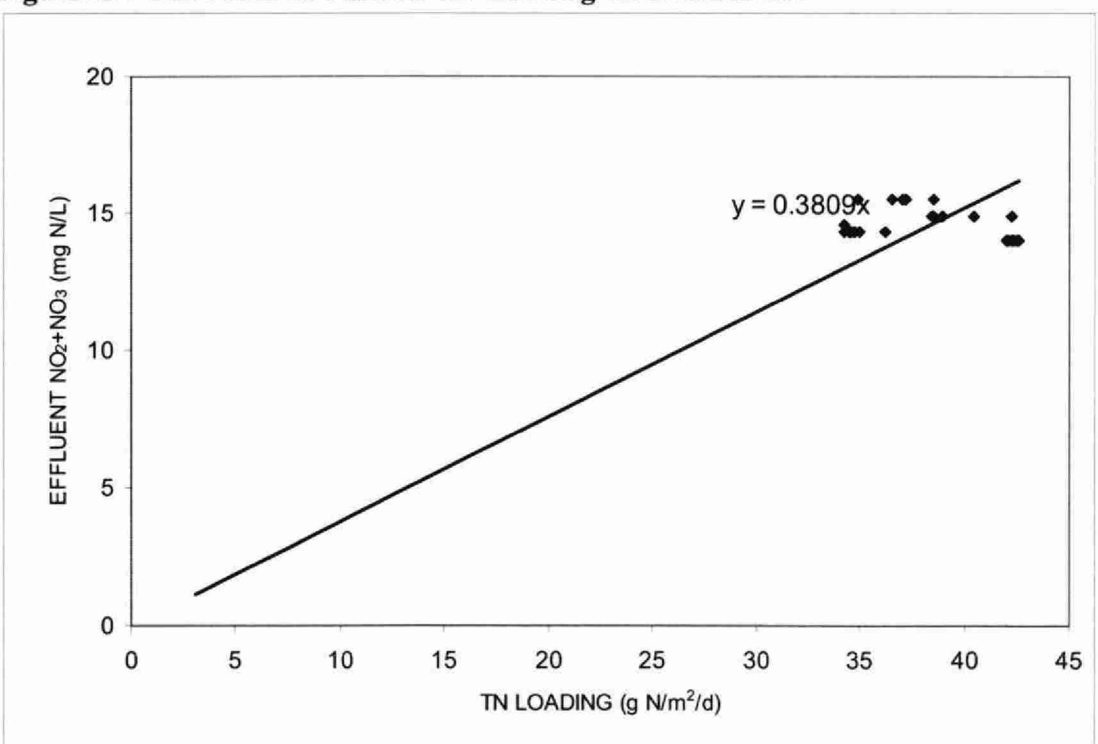

Figure A.V 3.20: RSF #3 Phase 3 TN Loading vs. Effluent $NO_2 + NO_3$

A.V 3.5 RSF #3 Phase 4

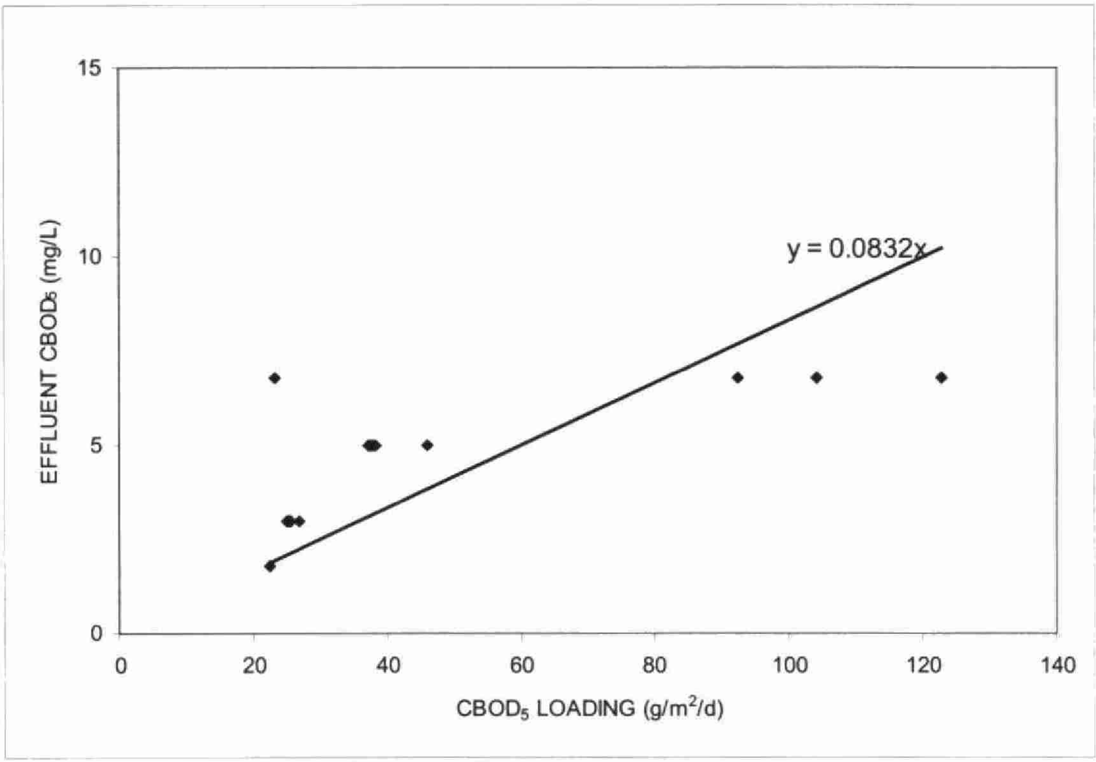

Figure A.V 3.21: RSF #3 Phase 4 $CBOD_5$ Loading vs. Effluent $CBOD_5$

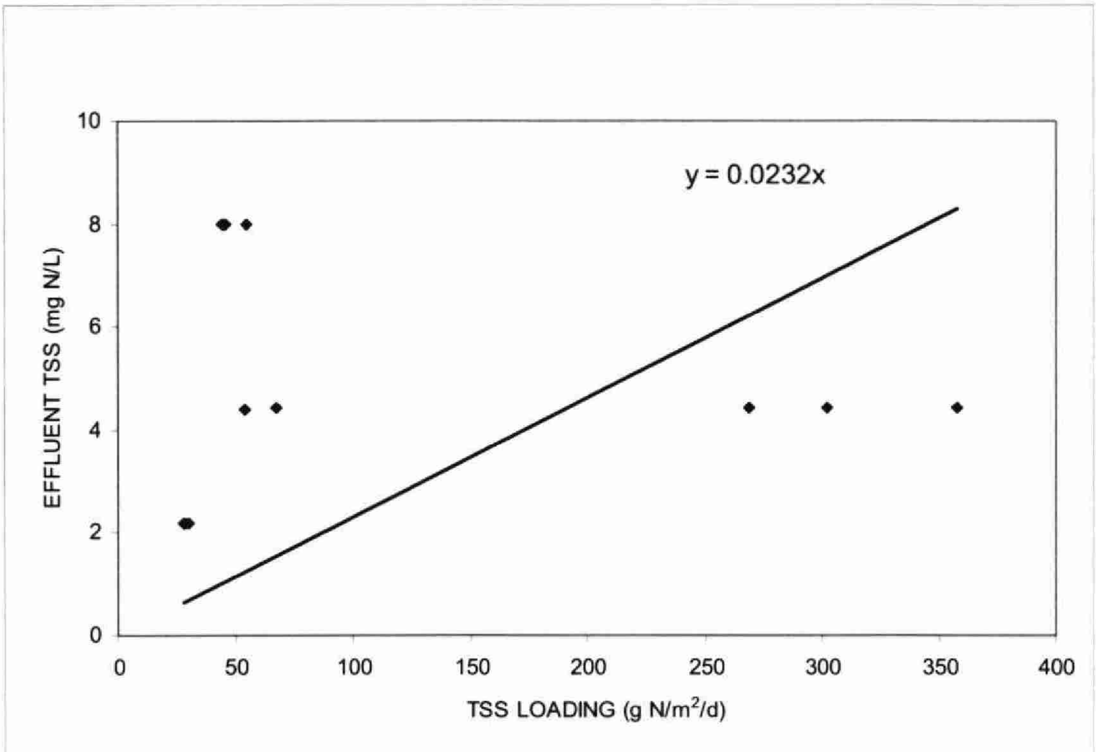

Figure A.V 3.22: RSF #3 Phase 4 TSS Loading vs. Effluent TSS

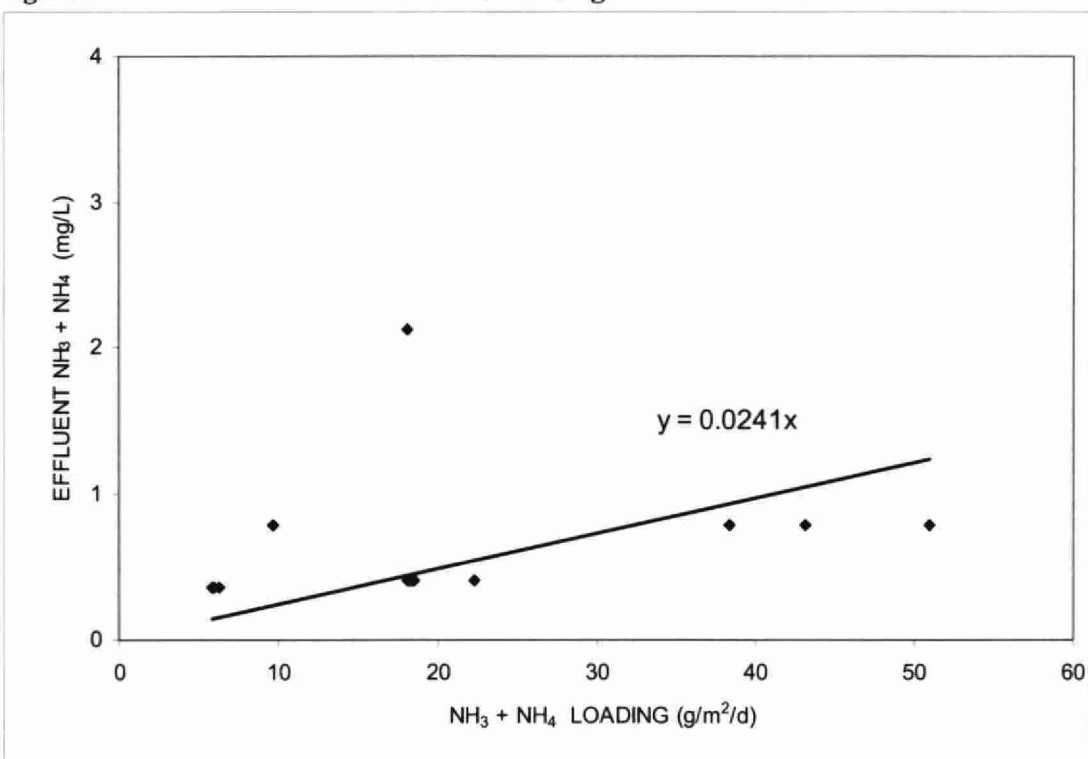

Figure A.V 3.23:RSF #3 Phase 4 NH_3 +NH_4 Loading vs. Effluent NH_3 +NH_4

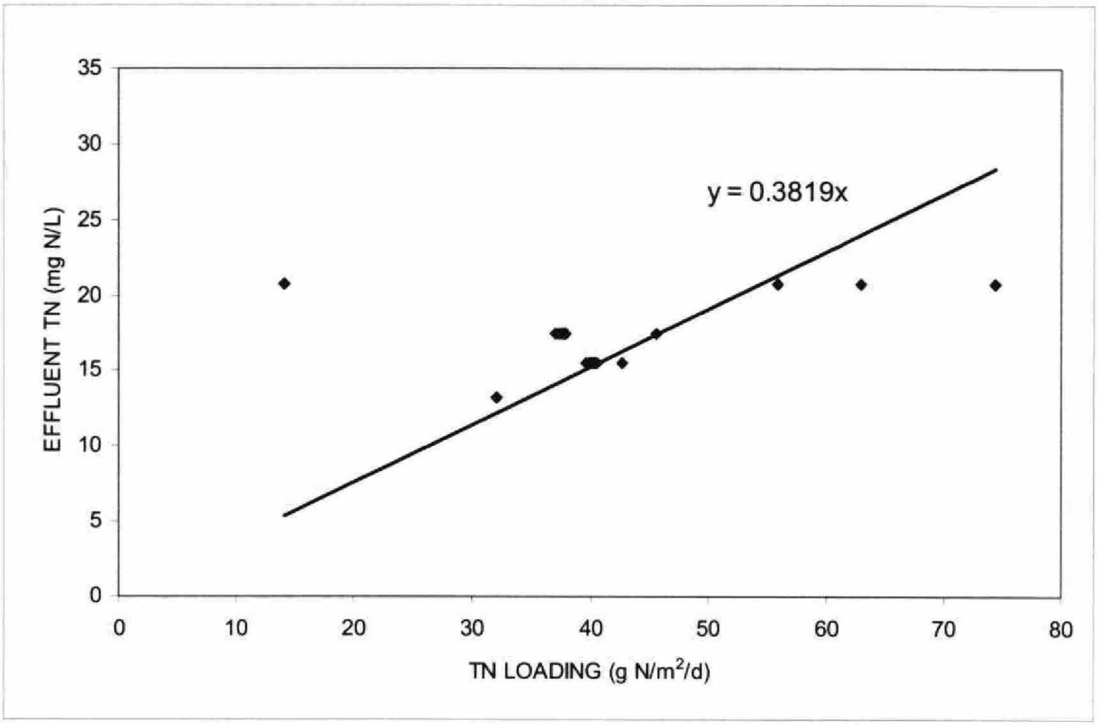

Figure A.V 3.24: RSF #3 Phase 4 TN Loading vs. Effluent TN

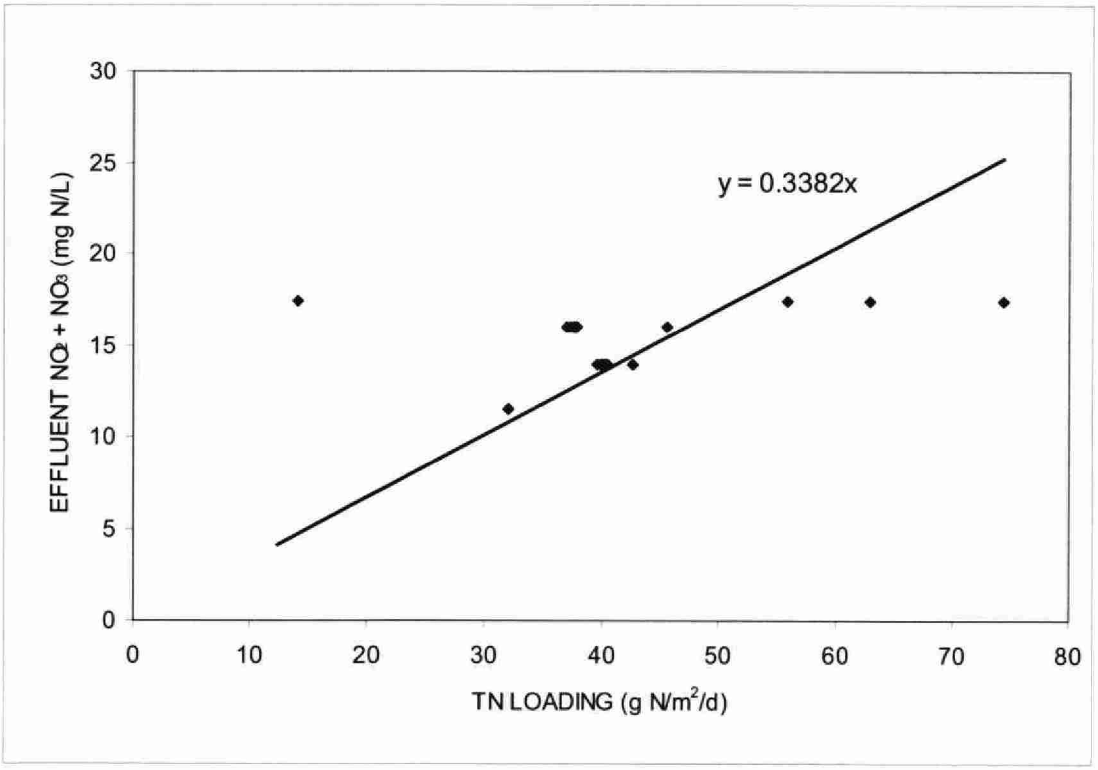

Figure A.V 3.25: RSF #3 Phase 4 TN Loading vs. Effluent $NO_2 + NO_3$

A.V 4.0 RSF #4 Loadings

A.V 4.1 RSF #4 Acclimation Phase

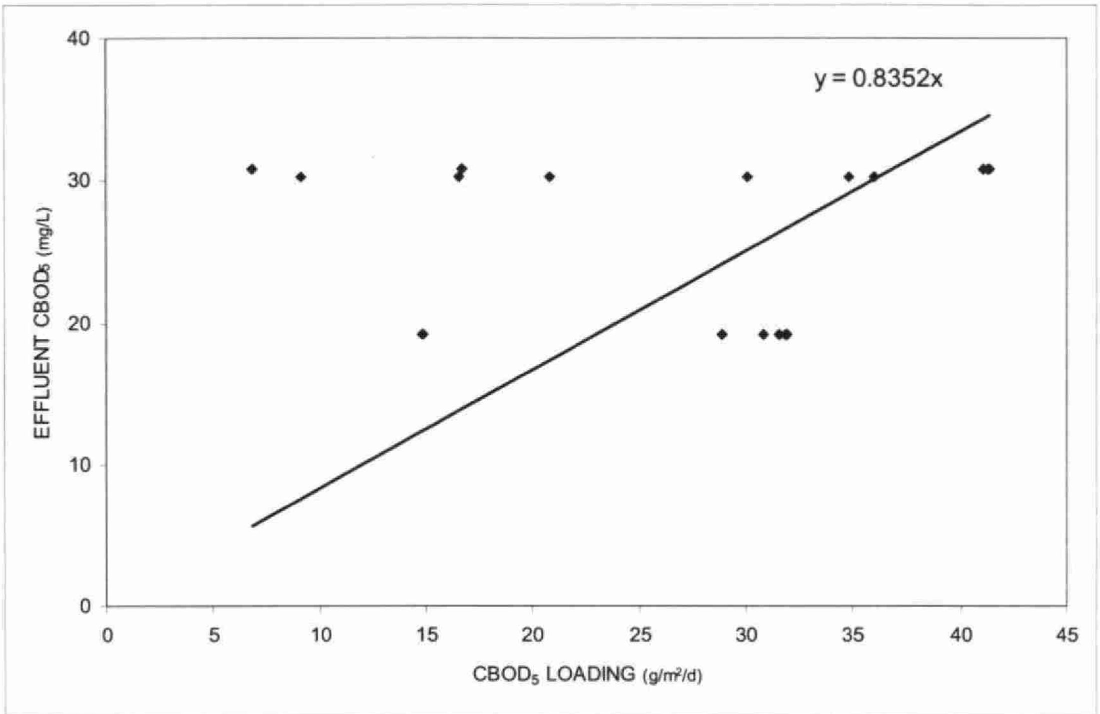

Figure A.V 4.1: RSF #4 Acclimation Phase $CBOD_5$ Loading vs. Effluent $CBOD_5$

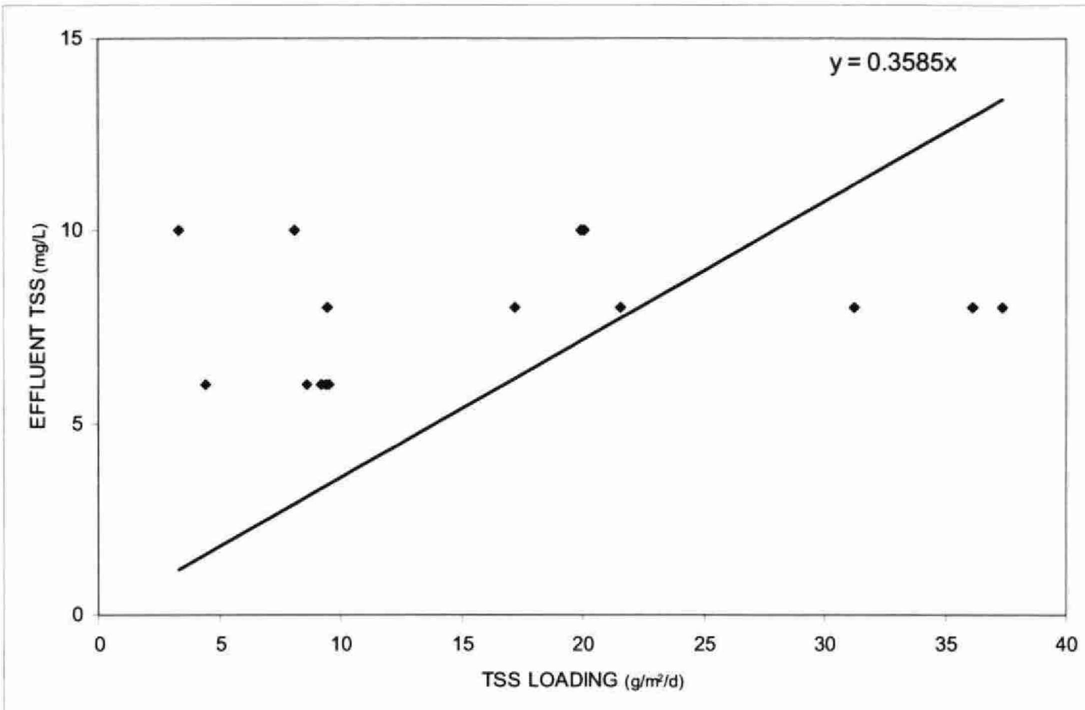

Figure A.V 4.2: RSF #4 Acclimation Phase TSS Loading vs. Effluent TSS

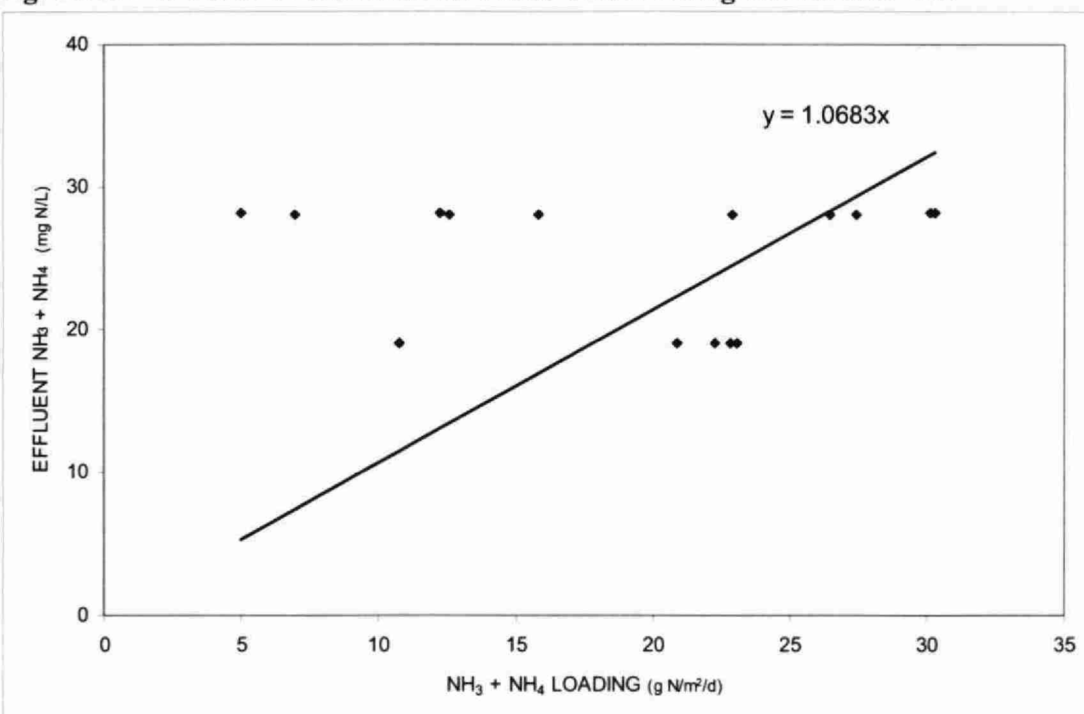

Figure A.V 4.3:RSF #4 Acclimation Phase NH_3 +NH_4 Loading vs. Effluent NH_3 +NH_4

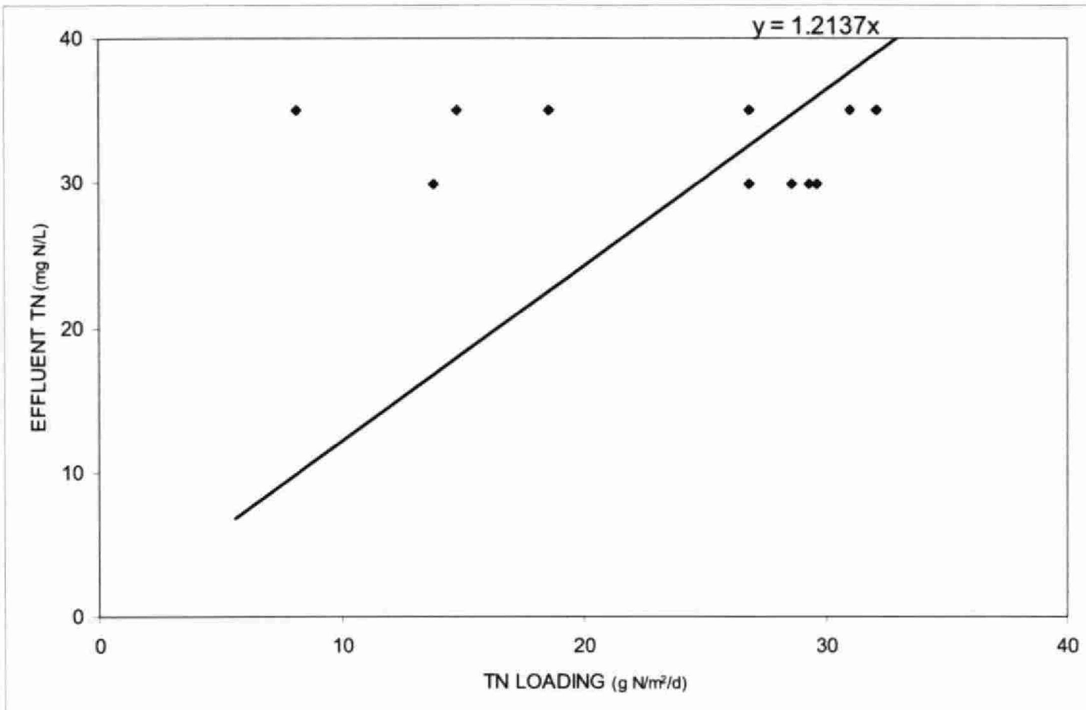

Figure A.V 4.4: RSF #4 Acclimation Phase TN Loading vs. Effluent TN

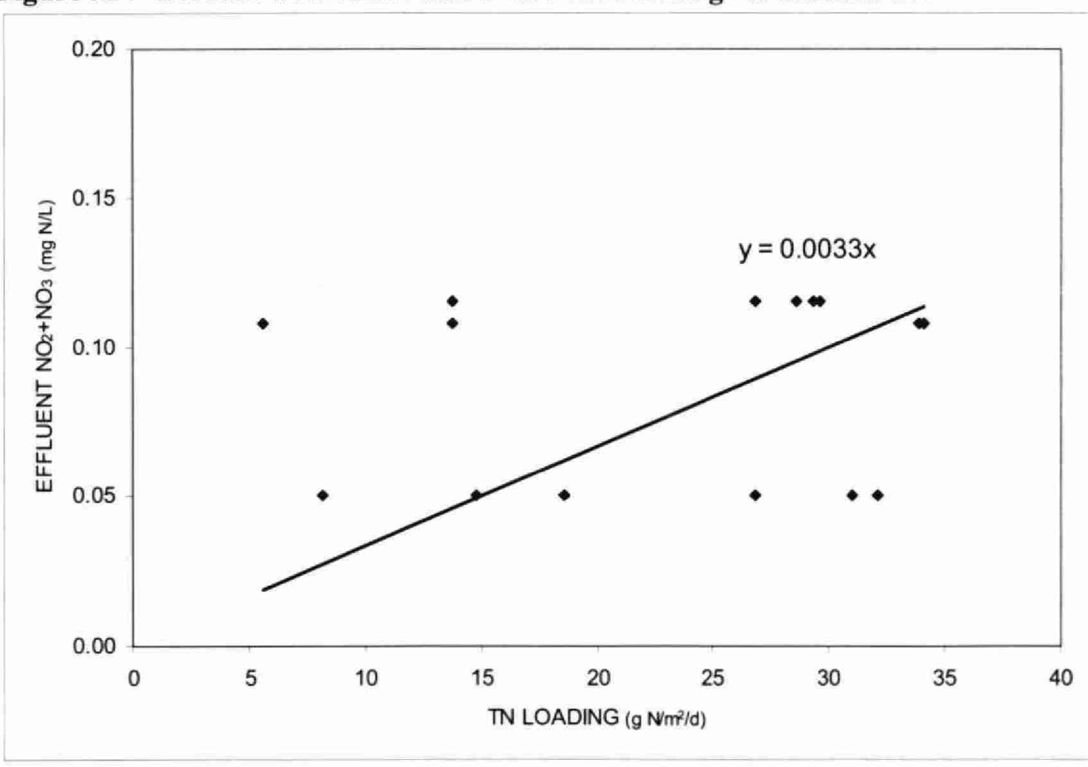

Figure A.V 4.5: RSF #4 Acclimation Phase TN Loading vs. Effluent NO_2 + NO_3

A.V 4.2 RSF #4 Phase 1

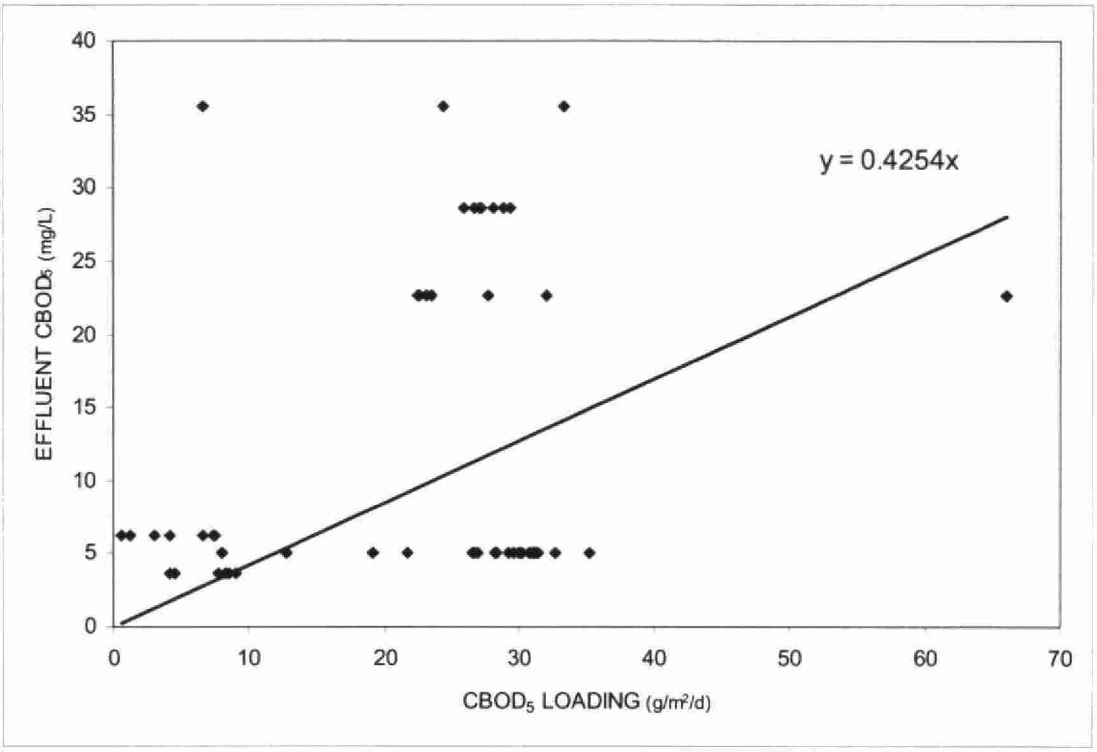

Figure A.V 4.6: RSF #4 Phase 1 $CBOD_5$ Loading vs. Effluent $CBOD_5$

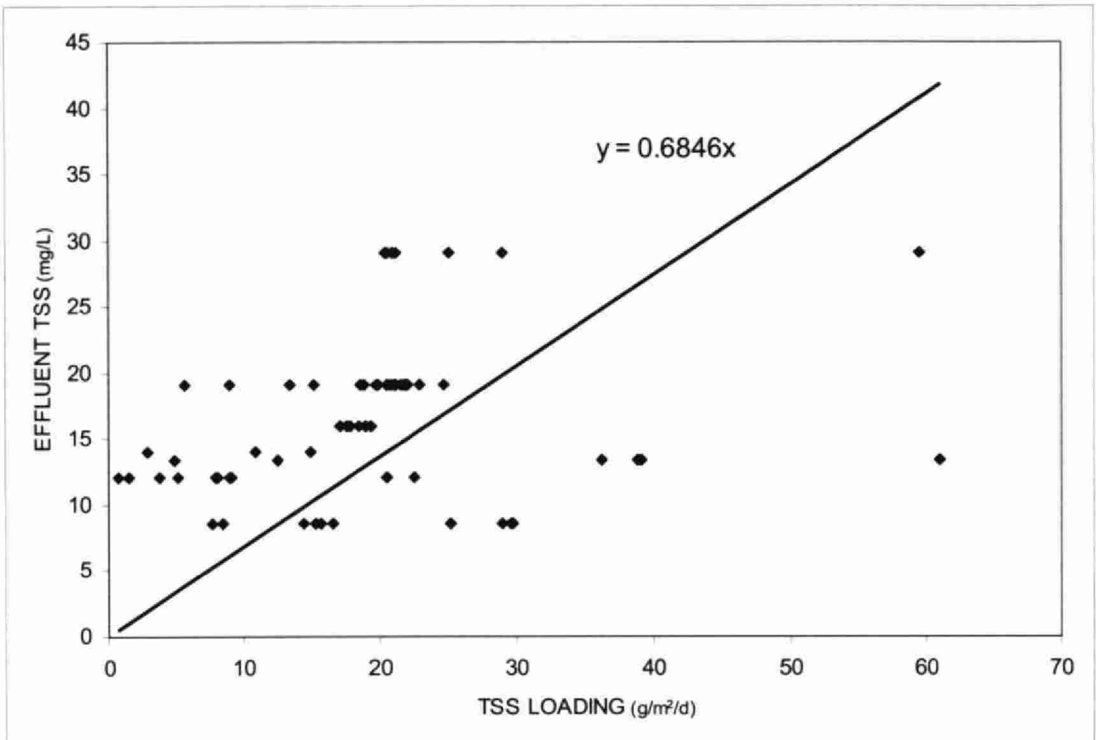

Figure A.V 4.7: RSF #4 Phase 1 TSS Loading vs. Effluent TSS

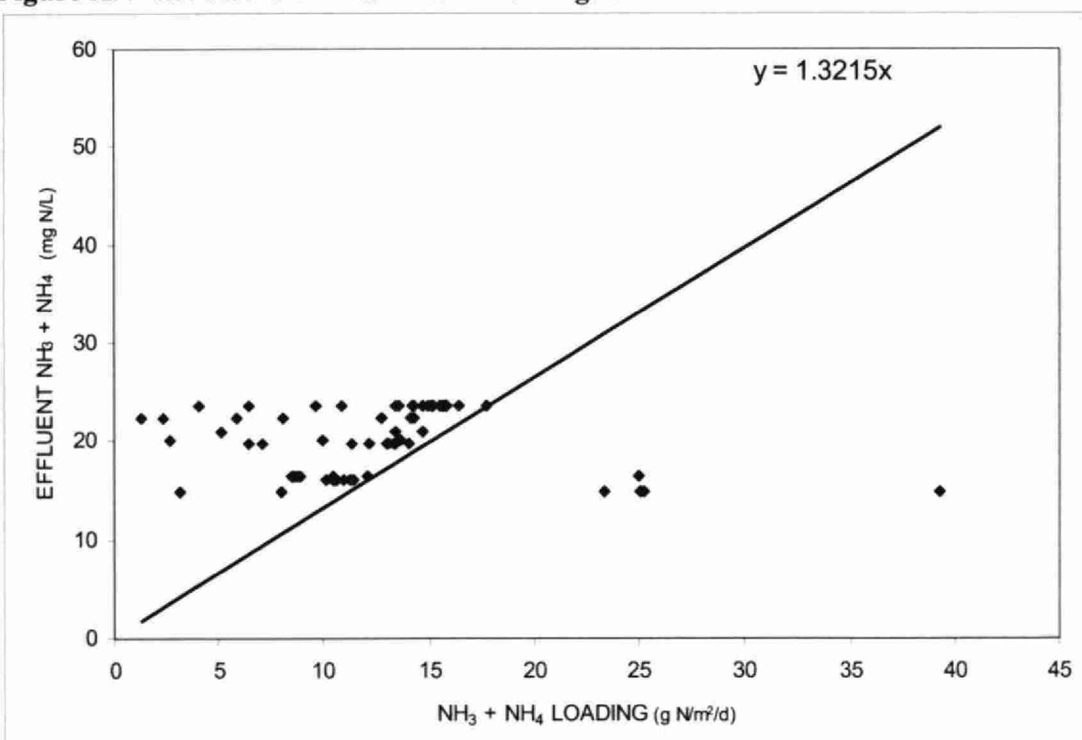

Figure A.V 4.8: RSF #4 Phase 1 NH_3 +NH_4 Loading vs. Effluent NH_3 +NH_4

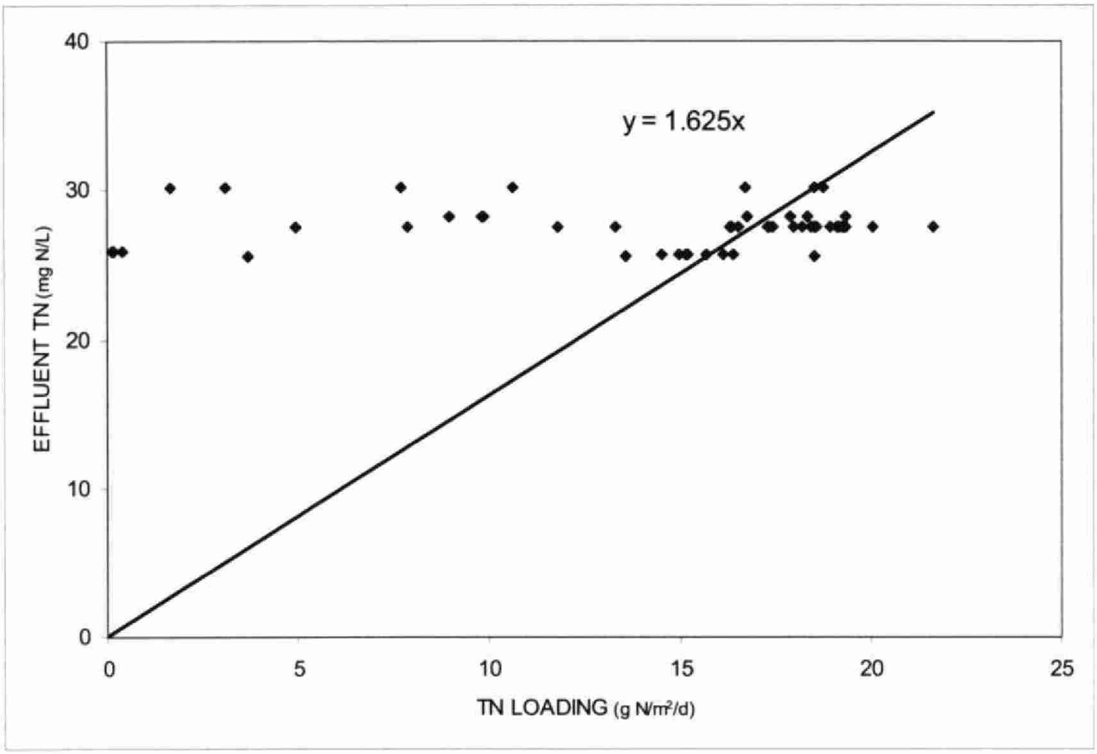

Figure A.V 4.9: RSF #4 Phase 1 TN Loading vs. Effluent TN

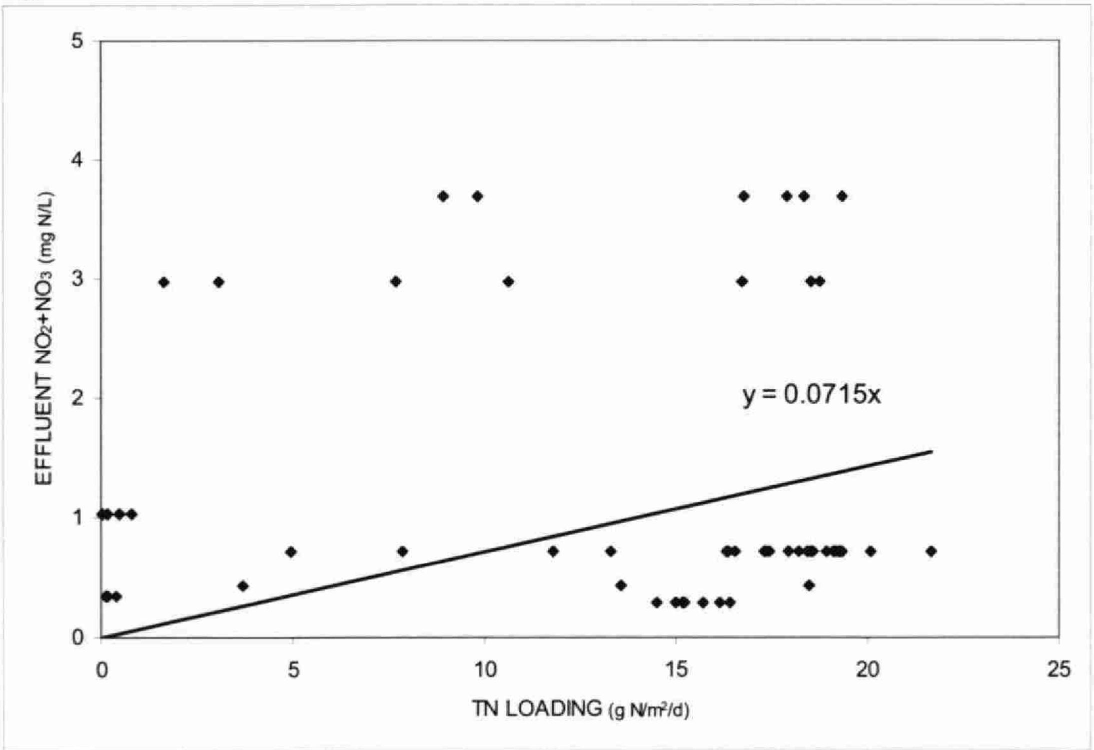

Figure A.V 4.10: RSF #4 Phase 1 TN Loading vs. Effluent NO_2 + NO_3

A.V 4.3 RSF #4 Phase 2

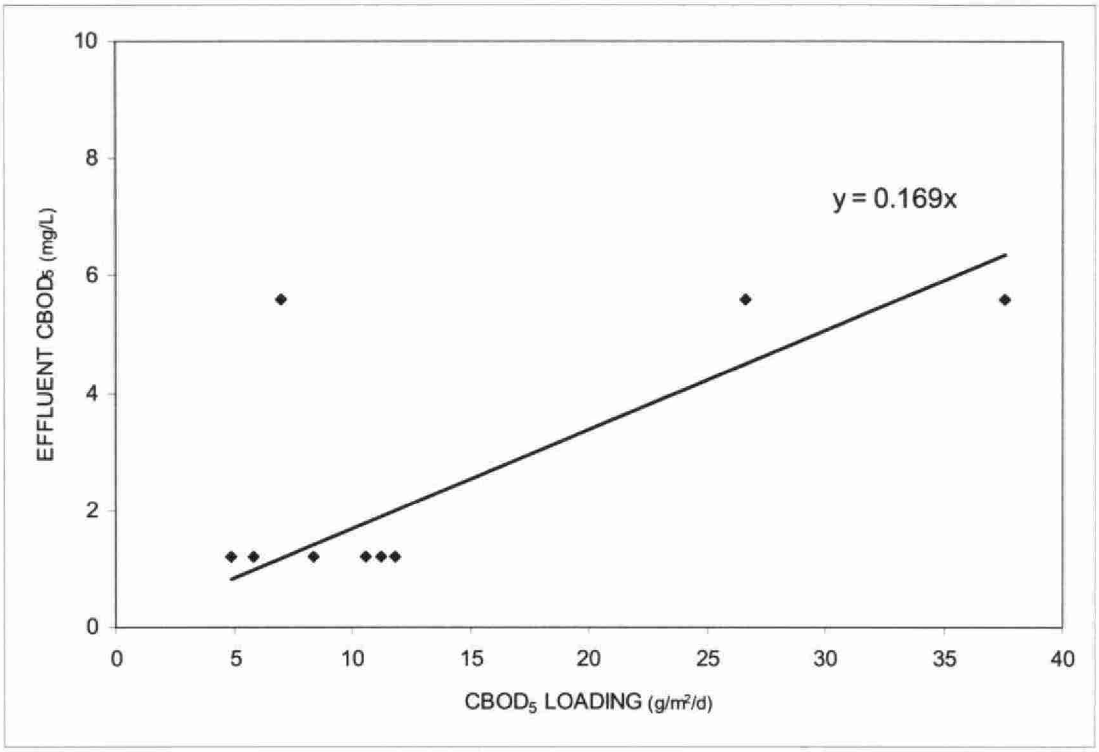

Figure A.V 4.11: RSF #4 Phase 2 $CBOD_5$ Loading vs. Effluent $CBOD_5$

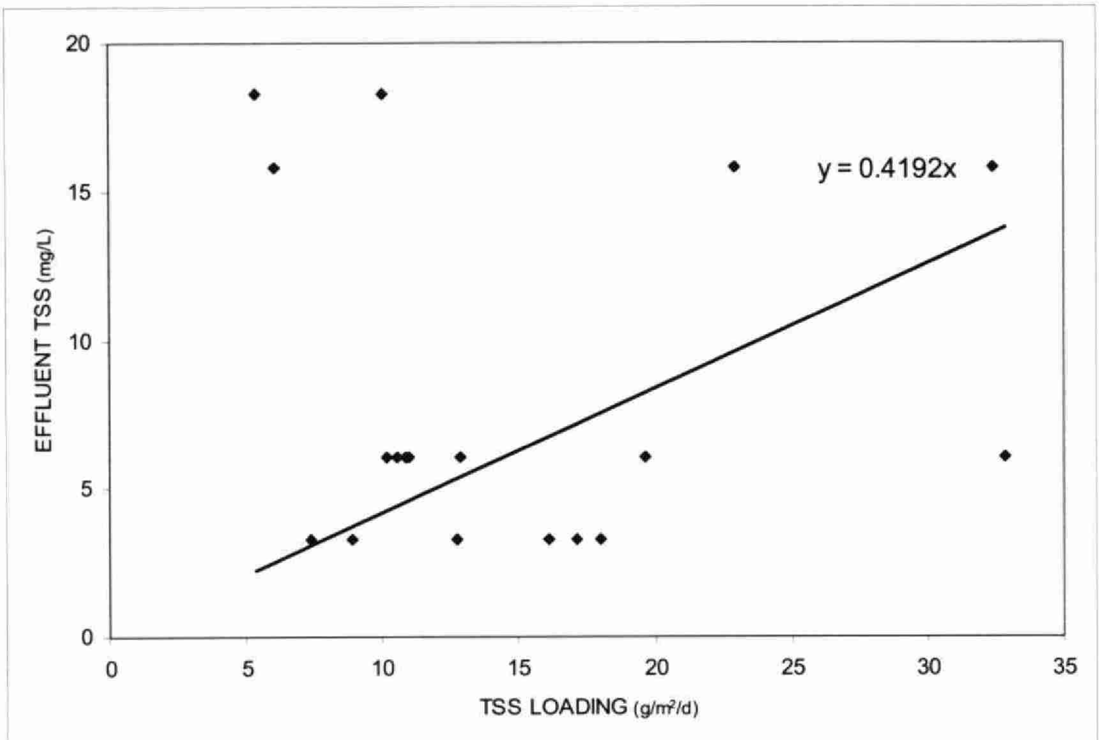

Figure A.V 4.12: RSF #4 Phase 2 TSS Loading vs. Effluent TSS

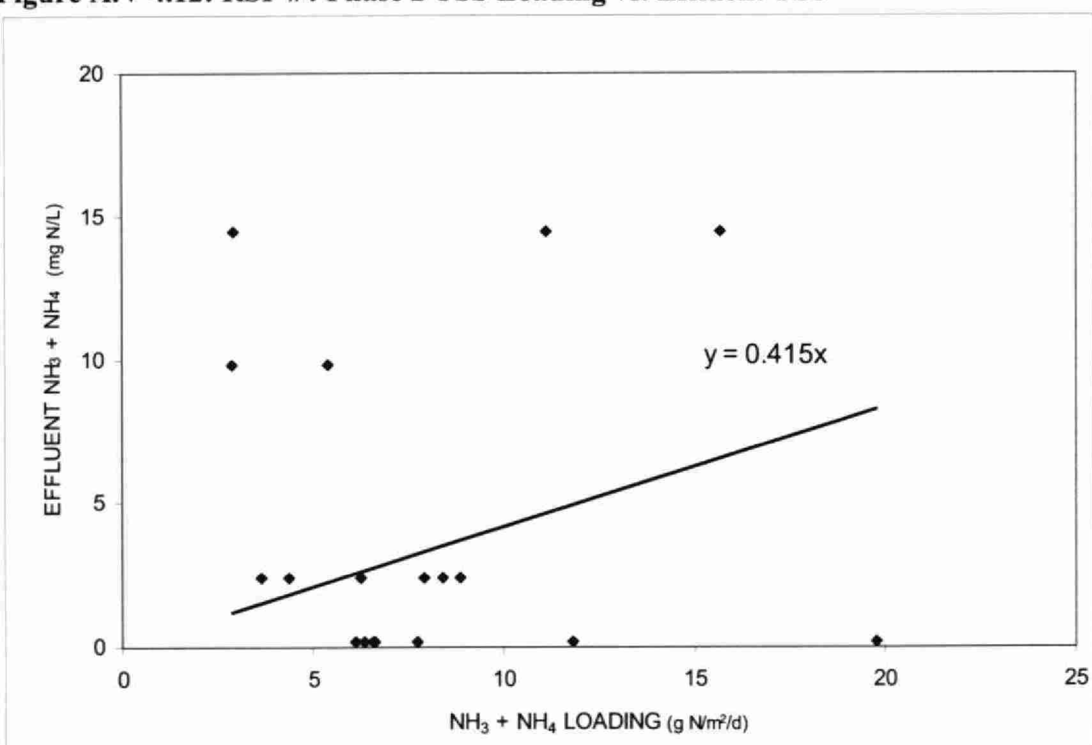

Figure A.V 4.13:RSF #4 Phase 2 NH_3 +NH_4 Loading vs. Effluent NH_3 +NH_4

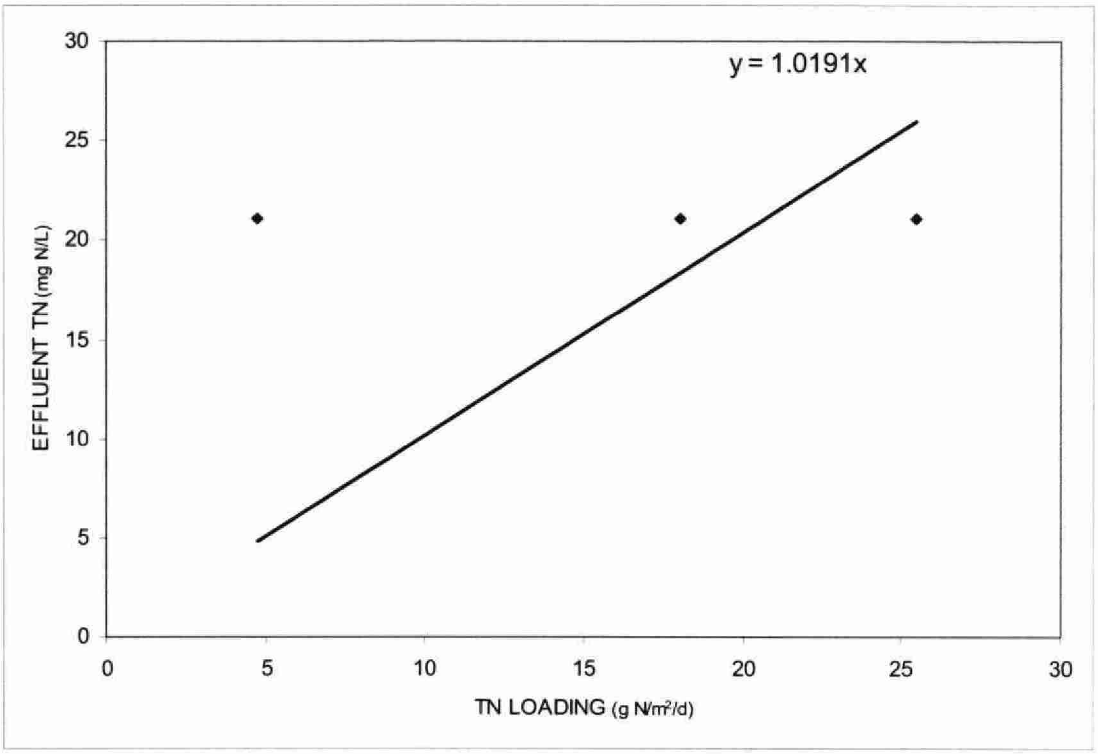

Figure A.V 4.14: RSF #4 Phase 2 TN Loading vs. Effluent TN

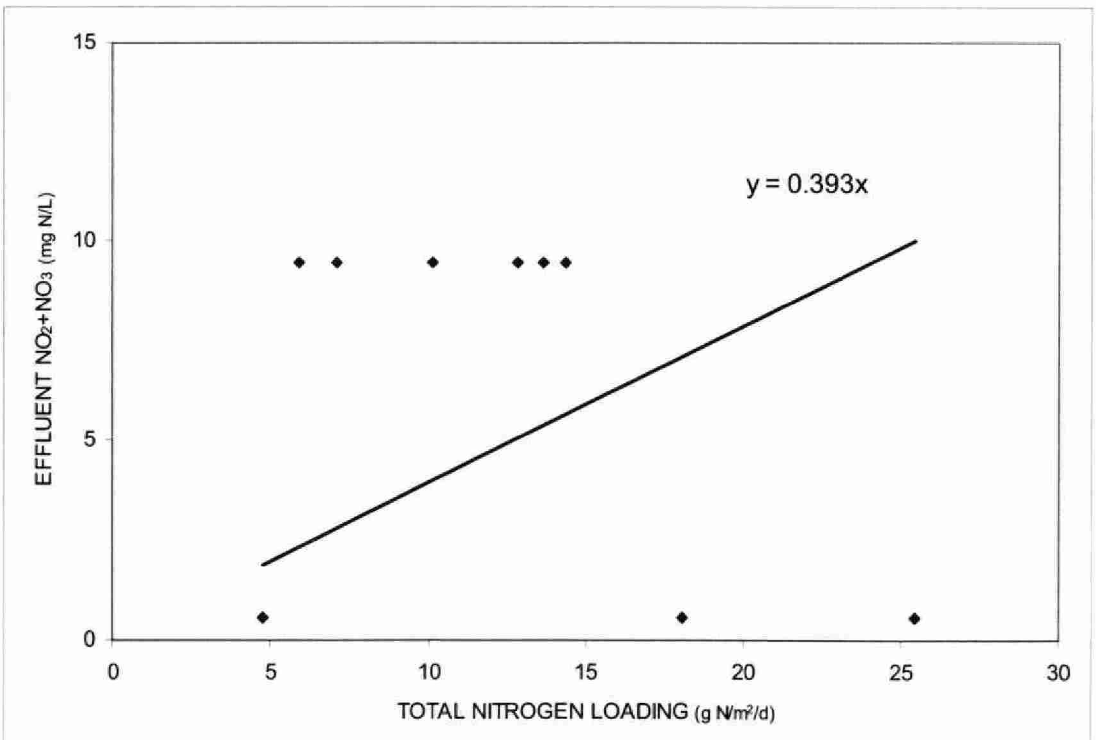

Figure A.V 4.15: RSF #4 Phase 2 TN Loading vs. Effluent $NO_2 + NO_3$

A.V 4.4 RSF #4 Phase 3

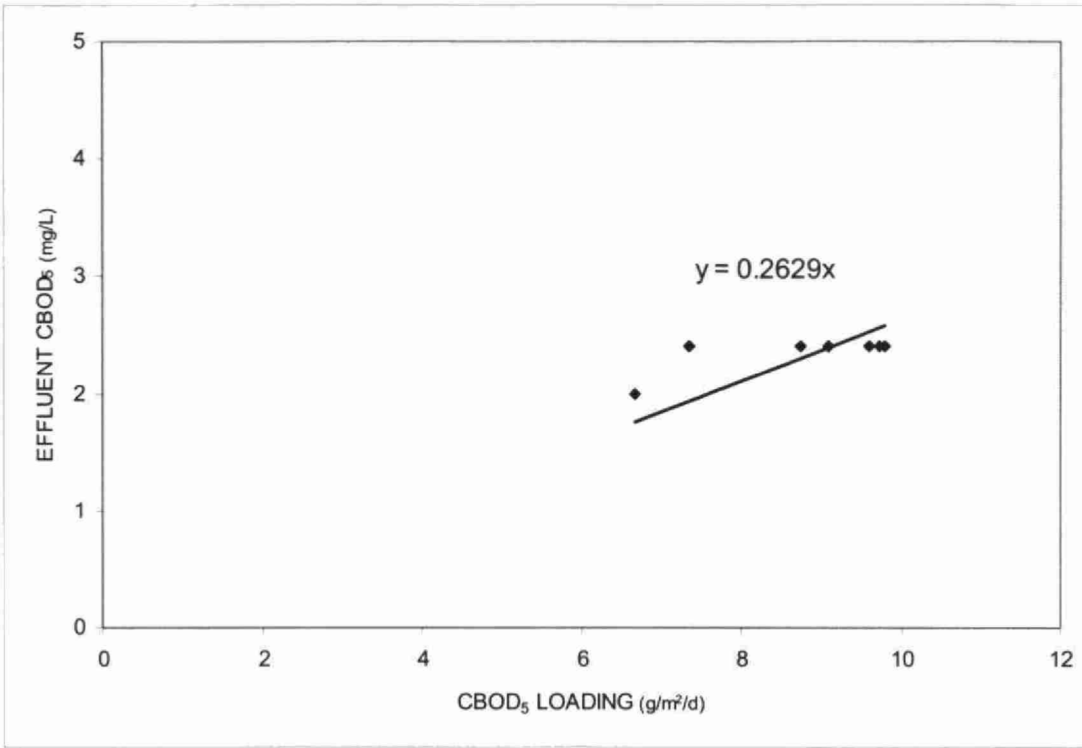

Figure A.V 4.16: RSF #4 Phase 3 $CBOD_5$ Loading vs. Effluent $CBOD_5$

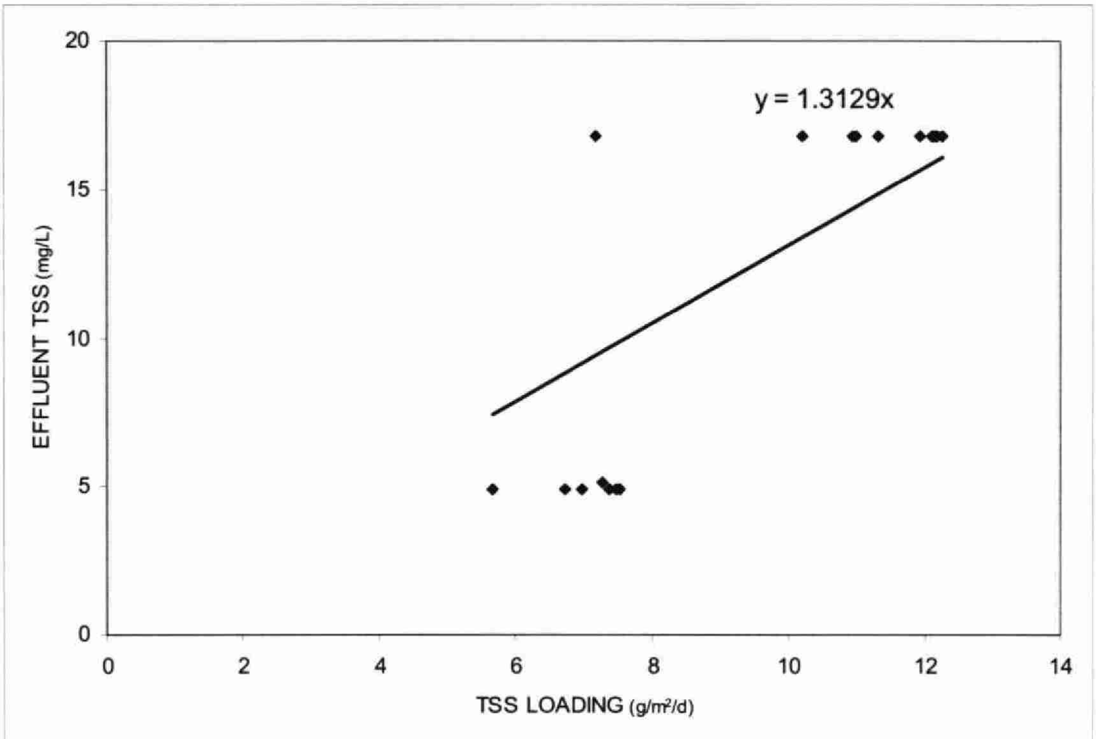

Figure A.V 4.17: RSF #4 Phase 3 TSS Loading vs. Effluent TSS

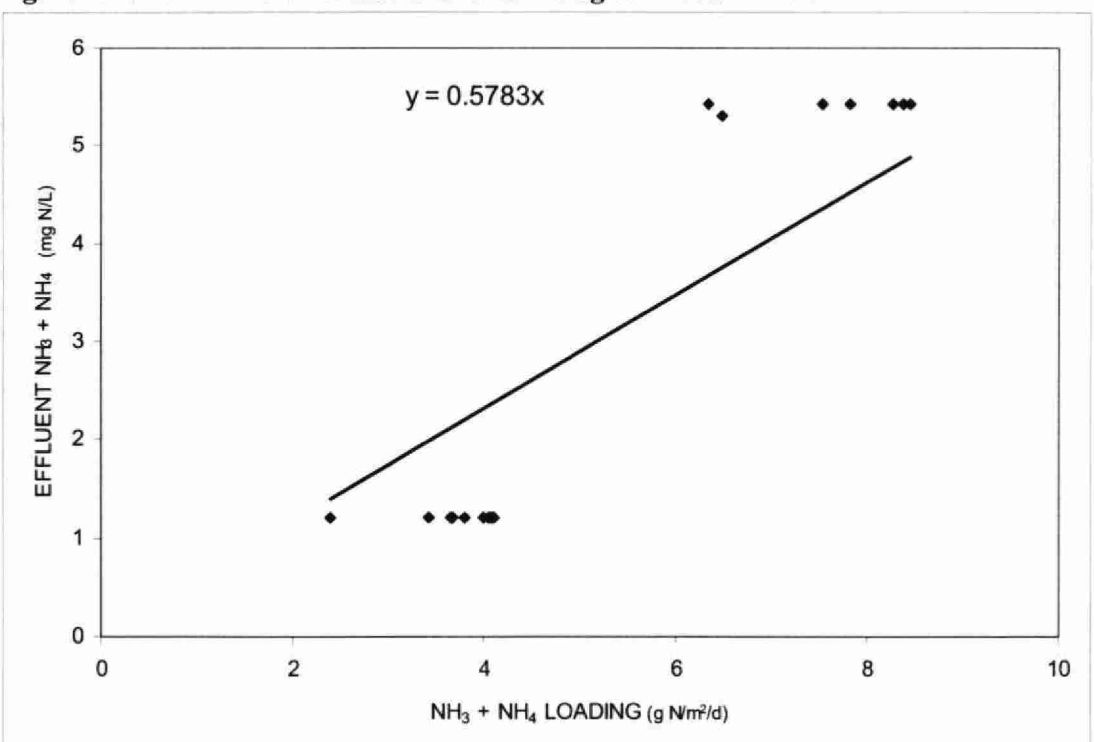

Figure A.V 4.18:RSF #4 Phase 3 NH_3 +NH_4 Loading vs. Effluent NH_3 +NH_4

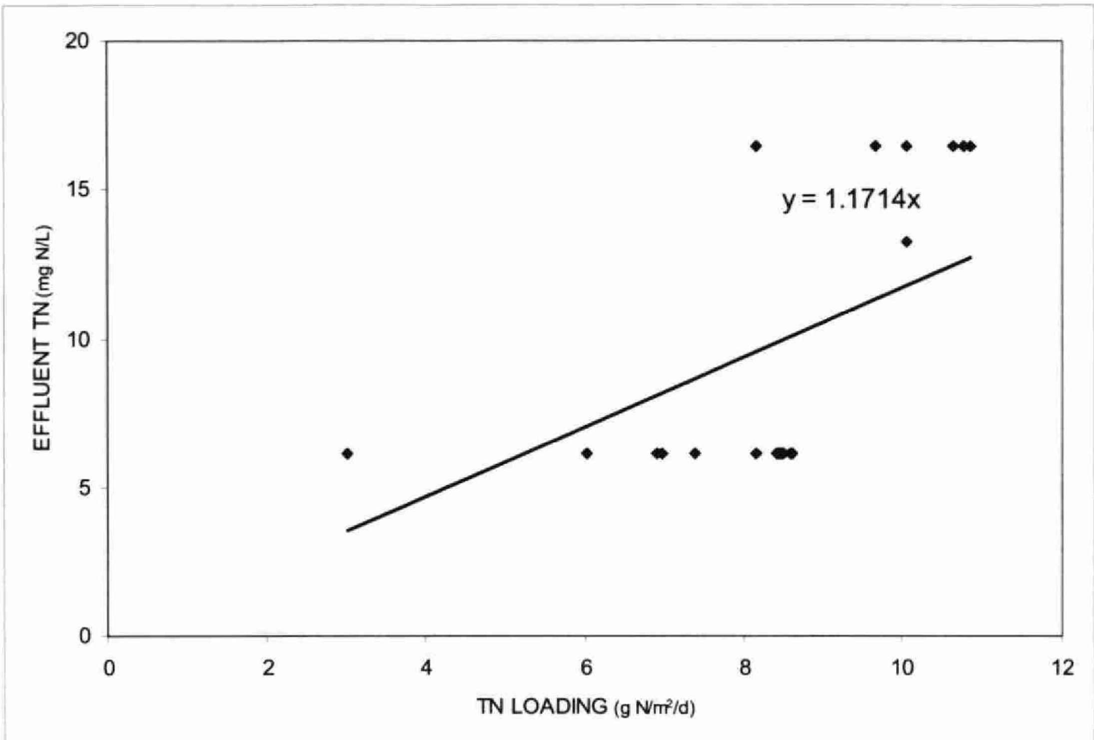

Figure A.V 4.19: RSF #4 Phase 3 TN Loading vs. Effluent TN

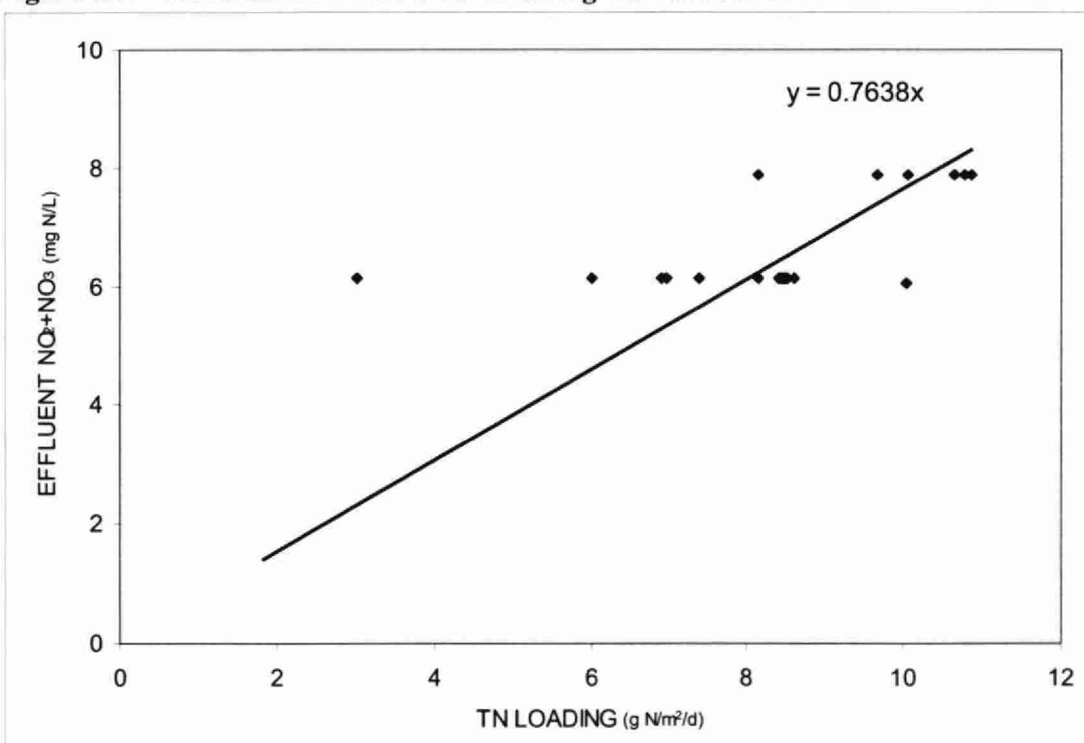

Figure A.V 4.20: RSF #4 Phase 3 TN Loading vs. Effluent $NO_2 + NO_3$

	DATE DUE		